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(INSTITUTED 1852.)

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ERRATA.

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Page 264, line 24: For "$2\frac{7}{8}$-in." read "two $\frac{7}{8}$-in."

Page 264, line 25: For "$2\frac{1}{2}$ x $2\frac{5}{16}$-in." read "$2\frac{1}{2}$ x 2 x $\frac{5}{16}$-in."

Page 264, line 27: For "$1\frac{7}{8}$-in. rivets" read "one $\frac{7}{8}$-in. rivet."

Page 264, line 29: For "$1\frac{3}{4}$-in. rivets" read "one $\frac{3}{4}$-in. rivet."

Page 402, line 33: For "fair" read "unfair."

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No. 842.

GAUGING OF CEDAR RIVER, WASHINGTON.

BY THERON A. NOBLE, M. Am. Soc. C. E.

PRESENTED OCTOBER 5TH, 1898

WITH DISCUSSION.

In the summer of 1895 the author was engaged by the Seattle Power Company to make the necessary plans, surveys, etc., for the installation of a water-power plant on Cedar River, Washington, the object being to transmit power to Seattle and Tacoma.

At this time no authentic information as to the discharge of the river was available, except an estimate made by Mr. Benezette Williams for the City of Seattle. Mr. Williams estimated that the minimum discharge was not less than 115 000 000 galls. in 24 hours, and that the maximum discharge was about 1 500 000 000 galls. in 24 hours, which would be equivalent to 177.93 and 2 320.8 cu. ft. per second, respectively.

As the minimum estimate by Mr. Williams, the accuracy of which was somewhat doubtful, would not allow for much future extension of the capacity of the proposed plant, and as the contemplated use of water for power purposes would take most of this discharge during extreme low water, the author recommended that Cedar Lake, which is about 15 miles above the intended point of diversion and contains

1 200 acres of storage area at low water, be used as a storage reservoir to increase the flow during the dry season.

To determine the minimum flow of the river and the storage capacity available in the lake, a series of discharge measurements was made at Clifford Bridge which crosses the river about 600 ft. below the proposed point of diversion and is 12 miles above the mouth. Since September 13th, 1895, a daily record of the gauge height has been kept at the same point.

Before a current meter was purchased, it was thought that these measurements could be taken with sufficient accuracy with floats, and the calculations made by Kutter's formula. For this purpose a place was selected about 900 ft. above Clifford Bridge, where the cross-section and flow are comparatively regular for a considerable distance. The bottom of the river here contains a good many boulders, ranging from 2 ft. to 6 ins. in diameter, and the current is slow and not broken up on the surface at ordinary stages of the river. The average cross-section, wetted perimeter, maximum velocity and slope were measured for different stages of the river, and the discharge calculated by Kutter's formula, using 0.035 as the value of n. Four of these discharge measurements were taken simultaneously with meter measurements at Clifford Bridge, which made it possible to calculate the true coefficient in Kutter's formula at these two points in the river where the character of the bed is very different. The results of these calculations are shown in Table No. 1.

The few float measurements that were taken indicate that at this point there is a radical difference in the value of the coefficient of roughness n at different stages of the river. The error due to assuming 0.035 as the value of n is 48% for low stages of the river. At the highest stage measured at this point the error is only 15%, and it is probable that at still higher stages, the error would disappear.

It will be interesting to note the value of n in Kutter's formula at the point of meter measurements, which was 900 ft. below the point of float measurements, and at which the character of the river is radically different. It was impossible to obtain the slope at this point, as the river, for about 200 ft. above, is very much more rapid, and the velocity of the current at the point of measurement must be influenced considerably by these rapids. The average slope, however, was taken for some distance above and below, to take in all

TABLE No. 1.—DISCHARGE OF CEDAR RIVER. MEASUREMENTS BY FLOATS.

DATE.	Average area of cross-section.	Mean hydraulic radius.	Sine of slope.	Assumed coefficient of roughness.	Actual coefficient of roughness.	Discharge calculated by float measurements.	Discharge calculated by meter measurements.	Ratio of maximum to minimum velocity.	Percentage of error.
1895.	sq. ft.	feet.		n.	n.	cu. ft. per second.	cu. ft. per second.		
August 19th...	208.6	2.431	.00025	.035	.0507	246	168	.47	47
September 13th	223.5	2.500	.00025	.035	.05122	269	182	.44	40
" 6th	218	2.466	.00025	.035	.04847	260	186	.47	48
" 24th	294	3.000	.00025	.035	.03991	408	356	.55	15

irregularities near the point of gauging. The value of n, however, is not materially affected by any possible error in the slope, a change of 100% in which would make no appreciable difference. The following is a list of the values of n at the different stages of the river, as calculated from Kutter's formula:

Discharge, in cubic feet per second.	Value of n.	Discharge, in cubic feet per second.	Value of n.
103.7.....	0.037	715.1.....	0.045
147.	0.029	808.1.....	0.042
165.7.....	0.029	1 302.7.....	0.031
169.7.....	0.030	1 549.9.....	0.030
185.7.....	0.031	1 979.7.....	0.026
187.7.....	0.030	2 175.6	0.0297
356.5.....	0.040		

At this point in the river n increases with the increase of discharge, up to about 700 cu. ft. per second, and then decreases with the increase of discharge. At 1 300 cu. ft. per second the value of n remains nearly constant and is about the same as the lowest value at the point of float measurements, when discharging 356 cu. ft. per second.

At Clifford Bridge the river-bed is a comparatively even sandstone formation with no obstructions, except two or three small boulders lodged near the north bridge pier. The river flows between two rock-filled log cribs, which form the bridge piers (Fig. 1, Plate I). There was some leakage around and through these piers at very high stages of the river, but practically none when the flow was less than 2 000 cu. ft. per second. The flow at very high stages, as given in the

diagrams and tables, is more or less in error on this account and also from the fact that when the discharge was greater than 2 176 cu. ft. per second, it was estimated. Neither of these causes, however, would materially affect the discharge below 2 000 cu. ft. per second.

At the point of float measurement the stream represents a very rough condition. The boulders form eddies and side currents which must seriously reduce the effective area of discharge at low water. It would be impossible to estimate the nature or extent of this influence, which is probably nearly a fixed amount, and its effect on the discharge becomes proportionately less, as the area of cross-section increases with the height of the stream; whereas in Kutter's formula it is a constant quantity. It is evident that any assumed value of n would not give equally correct results at different stages of the river or at different places in the river at the same stage. This discrepancy is more pronounced in rapid and irregular streams, and unless the channel is very regular and the surfaces very smooth, it is impossible to obtain accurate or reliable results by surface-float measurements. The irregularity of the current in rapid streams is strikingly illustrated in the diagram, Fig. 1, in which the velocity of the current at Station 10, at the point of meter measurements, at different stages and depths of the river, is plotted. In almost every stream there are numerous side currents, eddies and irregularities in the flow, which are caused by obstacles and the unevenness of the river-bed a considerable distance above the point of gauging, which would not be accounted for in the measurement of the cross-section, etc., at the point of gauging. It is possible that, by taking the cross-section and the slope measurements for a sufficient distance along the river-bed to develop more accurately the average condition of the river, the discharge would be found to more nearly follow Kutter's formula. This, however, would make the formula very cumbersome, in many cases practically impossible to solve, and would still leave the question of the value of n to be settled.

To facilitate taking the meter measurements and soundings for the cross-section, a platform was constructed and suspended from the bridge as nearly as possible at right angles to the direction of the current and as near to the surface of the water in the river as was safe. To the up-stream face of this platform was nailed a graduated board, 6 ins. x 1 in., the top edge of which was leveled off, and projected $\frac{1}{2}$ in.

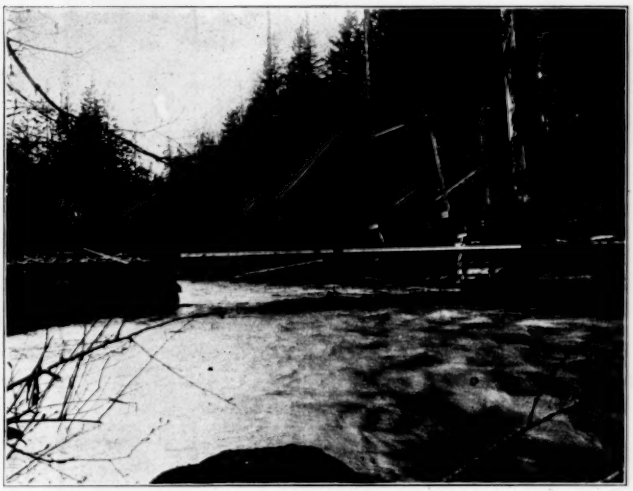

FIG. 1.

FIG. 2.

above the platform. From the top edge of this board all horizontal and vertical measurements were taken. A frame was constructed to hold the sounding rod and the meter vertically in the plane of the cross-section while the measurements were being taken. In a groove in the frame the rod or meter handle was free to move vertically in the plane of the cross-section, and the frame could be moved horizontally along the platform in the plane of the cross-section to any station desired. Fig. 1, Plate I, shows the operator with the meter and frame in position on the platform for taking velocity readings.

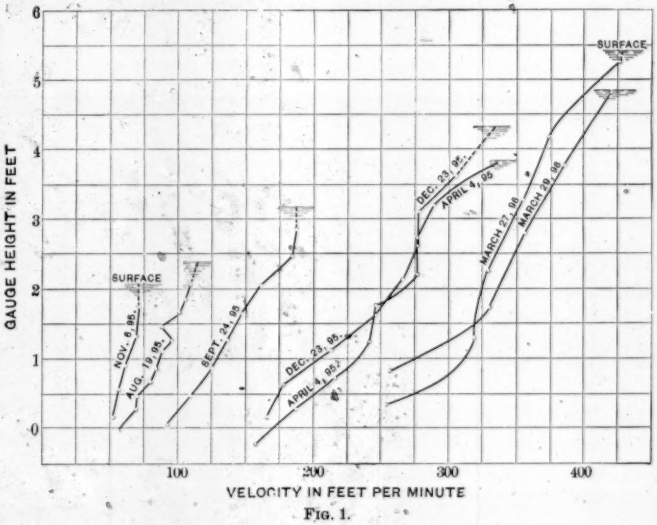

FIG. 1.

Haskell's current meter was used. The propeller wheel is 7½ ins. in diameter and is designed to make one revolution for every 2 ft. of current passing, the exact ratio being determined by rating the meter. The meter was rated twice during the discharge measurements, once over a 100-ft. course and once over a 200-ft. course. The 100-ft. course was measured on a No. 9 steel wire stretched between two piles in a favorable locality. For the 200-ft. course, the wire was stretched between the wharf and a pile 250 ft. distant. The place selected was well protected from the wind, and during the rating there was no appreciable current in the bay. Two buttons were soldered on the wire exactly 200 ft. apart. These points were used

as stations, and the stop watch in the velocity register was started and stopped while the meter in the boat was passing. The meter was fastened in a vertical position in a frame in the boat about 3 ft. ahead of the bow. The wheel was 2 ft. below the surface of the water. On each end of the boat screw-eyes were fastened, through which the wire ran and which acted as guides to keep the boat in line with the course and allow the meter to be pulled back and forth at any velocity desired. To attain high velocities a long rope was attached to the bow of the boat, and a man on the wharf pulled the boat along the course. On the 200-ft. course thirty-six velocity readings were taken, at various speeds, from 28 to 363 ft. per minute, and from these tests the meter was rated.

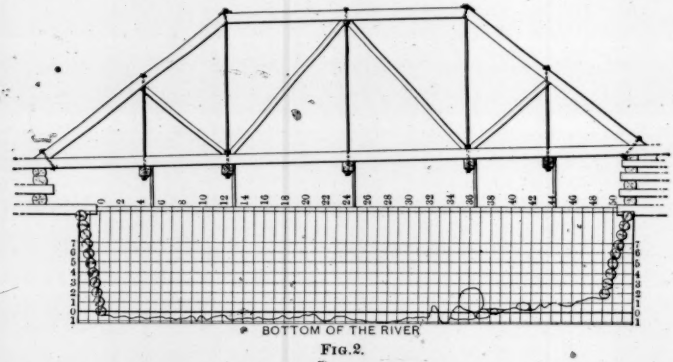

FIG. 2.

The cross-section of the stream at the point of measurement is shown in Fig. 2. Soundings were taken every 6 ins. across the stream. As the rod was always held in a vertical position in the plane of the cross-section, all the irregularities in the bed of the river at this point were accurately developed.

In taking discharge measurements the cross-section was divided into vertical sections 2 ft. wide and the full depth of the stream. In the middle of each of these sections, velocity readings were taken at depths varying from 0.2 ft. to 0.4 ft. apart (depending upon the height of the river) from the surface to the bottom. The velocity readings in each section were averaged and multiplied by the area of the section to obtain the discharge. The discharge of the river was obtained by

adding together the discharges of all the sections. In this manner discharge measurements were obtained for twelve different gauge heights ranging from 2.07 ft. to 6.26 ft., and representing from 104 to 2 176 cu. ft. per second. On account of the force of the current and the danger from floating logs, it was not safe to use the meter at higher stages of the river than the latter. The former measurement was taken at the lowest recorded height of the river, which occurred while a part of the flow was being stored in Cedar Lake during the construction of the storage dam at the outlet. These measurements are shown in Table No. 2.

TABLE No. 2.—Discharge Measurements by Meter.

Date.	Maximum velocity.	Average velocity.	Ratio of average to maximum velocity.	Area of cross-section.	Wetted perimeter.	Height on gauge board.	Discharge, in cubic feet per second.
1895.							
Aug. 10th...	141	96	.68	118.02	61	2.50	188
" 19th...	141	96	.67	105.72	60.5	2.40	170
Sept. 6th....	143	95.2	.67	116.64	61	2.49	186
" 24th....	209	140	.67	152.83	62.5	3.20	356
Nov. 6th....	105	67	.64	93.07	59.5	2.07	104*
" 14th....	131	90	.68	109.78	60.5	2.40	166*
Oct. 28th....	127	86	.68	108.37	60	2.27	1 470
Dec. 13th....	572	409	.72	318.92	70.1	6.26	2 176
" 23d....	348	220	.63	230.04	67.9	4.37	808
1896.							
March 26th..	544	403	.74	294.98	69.2	5.90	1 980
" 27th..	484	343	.71	271.40	68.5	5.42	1 550
" 29th..	452	318	.70	246.42	66.7	4.90	1 308
April 4th....	376	224	.60	191.78	64.2	3.90	715

* Taken when water was being stored in Cedar Lake.

From these measurements the stream was calibrated, giving the discharge in terms of the gauge height. The fluctuations in the discharge from August 10th, 1895, to July 1st, 1898, are shown in the diagrams Figs. 3 and 3 (*continued*). The monthly average is given in Table No. 3.

The greatest flow, 4 695 cu. ft. per second, occurred November 15th, 1896; the minimum natural flow, 138.3 cu. ft. per second, occurred October 31st, 1895. The maximum average monthly flow, 2 216 cu. ft. per second, occurred during December, 1896; the minimum average monthly flow, 163 cu. ft. per second, occurred during October, 1896. The average discharge from August 1st, 1895, to August 1st, 1896, was 846 cu. ft. per second; from August 1st, 1896,

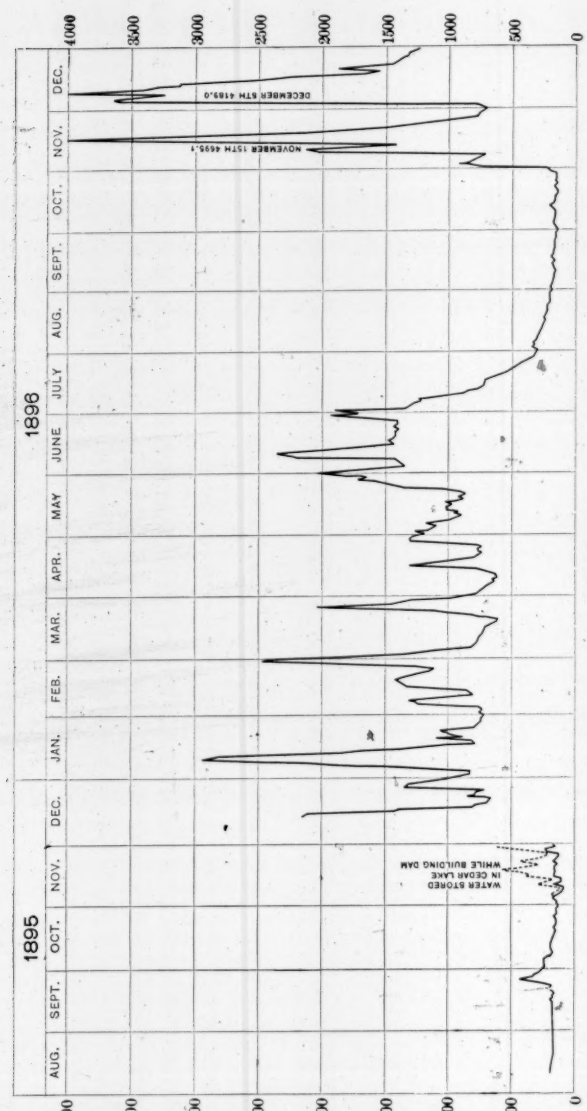
4000
3500
3000
2500
2000
1500
1000
500
0
1895
AUG.
SEPT.
OCT.
NOV.
DEC.
1896
JAN.
FEB.
MAR.
APR.
MAY
JUNE
JULY
AUG.
SEPT.
OCT.
NOV.
DEC.
NOVEMBER 15TH 4693.1
DECEMBER 6TH 4189.0
WATER STORED IN CEDAR LAKE WHILE BUILDING DAM

FIG. 3.

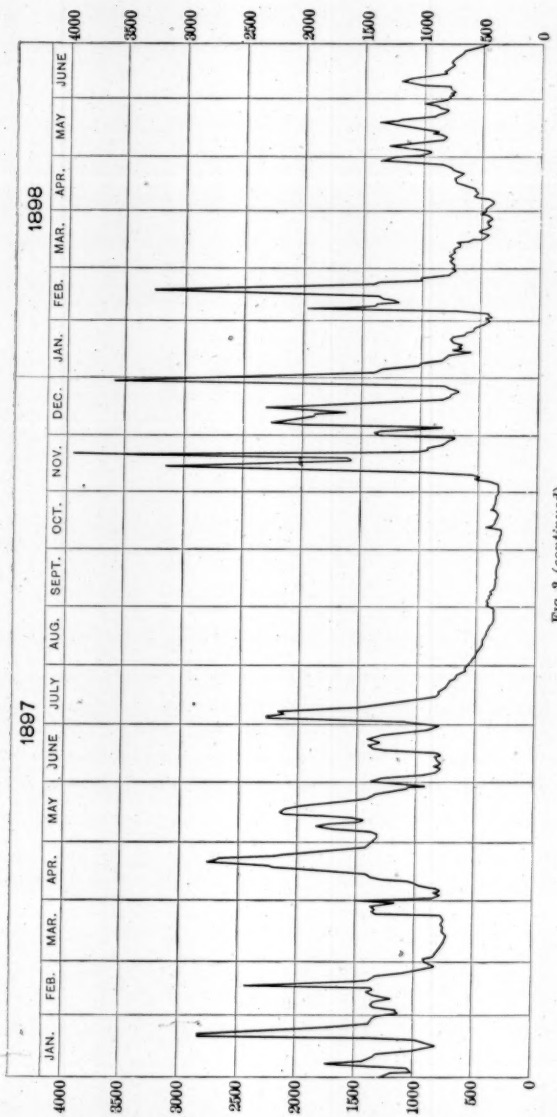

FIG. 3 (*continued*).

TABLE No. 3.—AVERAGE DISCHARGE AND RUN-OFF.

	1895.		1896.		1897.		1898.	
	Discharge cubic feet per second.	Run-off in inches.	Discharge cubic feet per second.	Run-off in inches.	Discharge cubic feet per second.	Run-off in inches.	Discharge cubic feet per second.	Run-off in inches.
January	.....	.....	1 318	10.56	1 454	11.42	850	6.47
February	.....	.....	1 339	10.06	1 302	9.45	1 377	10.60
March	.....	.....	939	7.53	798	6.40	587	4.71
April	.....	.....	788	6.13	1 599	12.43	716	5.56
May	.....	.....	1 284	10.32	1 562	12.55	998	8.01
June	.....	.....	1 653	12.85	1 057	8.22	801	6.23
July	.....	.....	733	5.89	1 135	9.12	.....	.....
August	193	1.55	260	2.09	448	3.60	.....	.....
September	225	1.75	229	1.78	350	2.72	.....	.....
October	172	1.38	163	1.31	339	2.72	.....	.....
November	397	3.09	1 594	12.40	1 319	10.26	.....	.....
December	1 110	8.92	2 216	18.16	1 637	13.15	.....	.....
Total	.....	16.69	.....	99.08	.....	102.04	.....	.....
Average	419.5	3.34	1 043	8.26	1 083	8.50	.....	.....

Drainage area of Cedar River = 143½ square miles.

to August 1st, 1897, 1 114 cu. ft. per second; from August 1st, 1897, to August 1st, 1898, 802.6 cu. ft. per second.

The drainage area of Cedar River has never been entirely surveyed. The upper part is in the heart of the Cascade Mountains. Two parallel ranges of mountains, spurs of the main Cascades, running in a westerly direction, form the valley of Cedar River. The mountains extend to within 10 miles of the mouth of the river on one side, and up to the mouth on the other side. Except near the summit of the Cascades at the head waters of the river, the mountains are heavily wooded with fir, cedar and hemlock. Between the summits of the two ranges the valley is from 3½ to 8 miles wide. The upper portion of the drainage area is high and precipitous, the mountains ranging from 5 000 to 8 000 ft. above sea level. Cedar Lake, which is about in the center of the drainage area, is 1 600 ft. above sea level. Fig. 2, Plate I, shows Cedar Lake looking east from the west end of the lake.

The portion of the drainage area below Cedar Lake varies in height from 20 to 5 000 ft. above sea level. The mountains in the lower portion of the valley merge into a series of high hills, which

are not precipitous. Lying between the mountains there is a considerable portion of level land, and benches which are very heavily timbered. The run off (see Table No. 3 and the diagram, Fig. 4) is calculated by assuming that the drainage area above Clifford Bridge is 143.5 square miles. This is correct except as to the upper or unsurveyed portion of the valley.

No direct information is available as to the precipitation in Cedar River valley. During 1896 a continuous record was kept at Monte

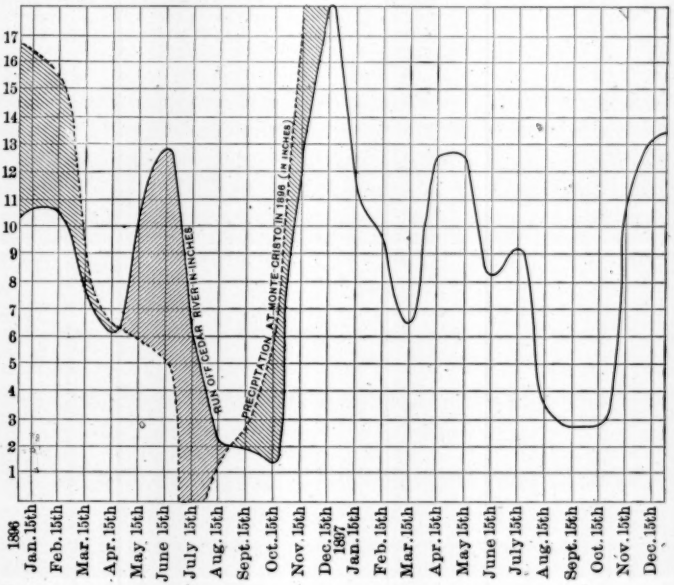

FIG. 4.

Cristo, which is 36 miles north of Cedar River, and occupies about the same relative position in the mountains as the center of the drainage area of Cedar River, and is subject to practically the same climatic conditions. In the diagram, Fig. 4, the monthly average precipitation at this point is plotted with the monthly run-off of Cedar River drainage area, to show the probable relation between precipitation and run-off in this region. During the winter months, while the snow is accumulating in the mountains, the precipitation is

very much in excess of the run-off; in the spring, while the snow is melting in the mountains, the run-off is much greater than the precipitation.

The amount of evaporation would be very difficult to determine, as the precipitation varies greatly in different parts of the basin, and innumerable stations would be required in order to get the average condition. It is not likely, however, that there is much loss by evaporation, as during most of the year the air in the mountains is near the point of saturation and the sky is overcast.

An interesting study of the influence of land surfaces on precipitation is afforded by the records of the signal stations in the Puget Sound region, between the summits of the Cascades and the Pacific Coast. In the fall, winter and early spring the prevailing winds are from the southwest, over the Pacific Ocean. At the sea coast, in the States of Washington and Oregon, they first come in contact with land surfaces, and the precipitation is enormous. At Cape Flattery and Clear Water, where the mountains reach down to the coast, it ranges from 100 to 130 ins. per annum. From the sea coast the lower air currents, in passing into the Sound country, follow the depressions between the ranges of mountains. The northern passage is through the Straits of Juan de Fuca, between the Olympic Mountains on the south and Vancouver Island on the north, and the air currents coming from the ocean over this course pass in succession in an easterly direction the following signal stations: Clear Water, Tatoosh Island, Pysht, Port Angeles and Coupeville, the latter being the center of the Sound region. In following the southern course the air currents first strike the coast between the mouth of the Columbia River and Gray's Harbor, taking a northeasterly direction around the southern end of the Olympic Mountains, and then in a northerly direction between the Cascades and the Olympic Mountains, passing in succession the following stations: South Bend, Olympia, Tacoma, Seattle and Coupeville. All these stations are at sea level and the difference in precipitation cannot be accounted for by difference in elevation. In Table No. 4 it will be noticed that the precipitation almost invariably increases from the sea coast to the center of the Sound region, at Coupeville, along the courses mentioned. From the coast of the Sound east toward the Cascades the precipitation rapidly increases again and apparently reaches its maximum at the summit. The actual

TABLE No. 4.—PRECIPITATION IN THE PUGET SOUND REGION.
MEASUREMENTS IN INCHES.

1897.	Clear Water.	Tatoosh Island.	Pysht.	Port Angeles.	Coupeville.	South Bend.	Olympia.	Tacoma.	Seattle.	Coupeville.	North Bend.	Discharge of Cedar River.
January....	16.52	12.20	8.31	8.77	2.79	8.49	5.81	6.92	3.74	2.79	10.07	11.42
February...	.....	11.16	5.19	2.17	1.82	10.30	6.58	5.19	2.99	1.82	10.05	.....
March	.....	11.31	7.86	2.40	3.49	11.24	6.29	4.73	3.05	3.49	7.65	.....
April	.....	7.61	2.75	.48	.88	3.61	1.28	1.64	1 53	.88	5.40	.....
May	.....	3.45	1.88	.03	2.18	2.58	1.02	1.46	1.30	2.18	3.86	.....
June.....	.....	3.37	1.87	1.16	.84	3.77	1.49	1.54	1.67	.84	5.33	.....
July.....	.....	3.11	.90	.88	1.46	2.19	.89	.87	2.36	1.46	3.84	.....
August.....	.....	2.66	1.36	.28	.85	.86	.36	.58	.24	.85	Not given	.....
September..	3.15	2.16	1.54	1.32	1.37	2.65	2.62	1.41	2.04	1.37		.....
October	7.40	2.91	2.55	1.13	1.80	3.43	1.55	1.63	1.92	1.80		.....
November..	23.85	17.77	14.37	6.03	4.42	18.68	13.01	12.31	8.89	4.42	22.07	.....
December..	23.66	17.50	18.98	9.85	3.97	20.70	17.60	14.48	11.80	3.97	22.62	.....
Total....	.....	95.21	68.28	28.10	25.87	89.49	58.50	52.76	41.53	25.87	.....	.....
1895.....	.....	92.95	.....	24.07	.....	.....	.....	37.05	29.69	.....	.....	.....
1896.....	*102.20	100.85	.....	30.42	22.86	.....	65.46	51.02	42.83	22.86	.....	.....
1897.....	.....	95.21	68.21	28.10	25.87	89.49	58.50	52.76	41.53	25.87	90.00	.....

* Eleven months, January not included.

run-off from the drainage area of Cedar River, allowing for the loss by evaporation and the decrease of precipitation toward the lower portion of the drainage area, is greater than at any station in the Sound region at which records are kept. At the summit of the Cascades the precipitation must be very great, possibly as much as 200 ins. per annum.

It is a common belief that there is some more or less well-defined relation between the amount of rainfall and the elevation of the locality above sea level. The record of precipitation in Puget Sound is a striking illustration of the fallacy of such a supposition. At Clear Water, on the sea coast, the precipitation is greater than at Monte Cristo, which is 2 700 ft. above tide water, in the heart of the mountains directly east of Clear Water. Though precipitation in most cases is greater at higher elevations than in the adjacent valleys, it would not be safe to apply this rule to all possible combinations and climatic conditions. It is very doubtful if there is any relation that would be the same for different localities or for different periods of time in the same locality. The increase of precipitation in the higher elevations

is largely influenced by the fact that the upper air currents pass over the intervening land surface high enough to be out of reach of its influence and reach the mountains in a saturated condition, thus causing the great precipitation.

The question of precipitation is not generally of much value to the practicing engineer, because of the many influences governing its amount and variation in different localities, and the impossibility of securing data which will give a reliable basis for the calculation of the actual discharge of a stream. The only excuse for touching on the subject in this paper is to bring out some features peculiar to the region about Puget Sound, which are unusual, and will perhaps be interesting as indicating the futility of attempting to establish any relation between the run-off and the precipitation.

It behooves the engineer whose work depends upon the value or character of the discharge of any stream, to determine by actual measurement the fluctuation of discharge covering a sufficient period of time to develop any unusual conditions that are liable to occur, and the longer the period of time the observations cover, the greater their value. In adjacent drainage basins, where the topography of the country and the climatic conditions are practically the same, the discharge will be subject to the same fluctuations, and it is likely that the amount of discharge will be nearly proportional to their relative drainage areas. This is quite clearly shown in a paper by C. C. Babb, Jun. Am. Soc. C. E., * in which the average monthly discharge of the drainage areas of the Croton and Sudbury rivers are plotted on the same diagram. Even this relation, however, would not hold good if their respective areas were too large to be subject to the same climatic influences.

In reference to the discharge of rivers and streams, a most valuable service is being rendered to the engineering public by the United States Geological Survey, in collecting and publishing the results of discharge measurements of numerous streams in the United States, made by the department and by others.

No record of the discharge of a stream is of much value unless it covers a series of years, but it is very seldom that the development of an enterprise can be postponed for the length of time required to obtain the data upon which the most important features of the scheme

* *Transactions*, Am. Soc. C. E., Vol. xxviii, p. 331.

may depend. The result is that if the engineer cannot get the desired information from some reliable source, he is compelled to make estimates, and in these he can have but little confidence, for some unexpected flood or dry season is sure to come sooner or later and entirely upset his calculations and possibly cripple the value of the plant. It seems particularly fitting that the government should undertake and carry out this work, as the case is very exceptional where the engineer is in a position to take observations covering the long time necessary to secure data of much practical value.

DISCUSSION.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—When it is realized that the country of which the author writes has been but recently settled, and that very few observations regarding rainfall and stream-flow have been recorded, it is somewhat remarkable and gratifying to note that an engineer, Mr. Benezette Williams, when asked perhaps seven or eight years ago to furnish a plan for providing the city of Seattle with water, came so near to estimating the actual minimum flow of the Cedar River, as it has now been determined by gauging. This shows what an engineer in a new country can do toward getting comparatively near to the actual facts, when the true value and importance of the few available data are carefully scanned, weighed and applied to the locality where the new work is to be done, largely on the basis of the relation between the stream-flow and rainfall in other parts of the country where this relation is more accurately known. Mr. Williams estimated the minimum flow of the Cedar River, in round numbers, at about 178 cu. ft. per second, while the gaugings of the author later determined it, as the average monthly discharge for October, 1896, to be 163 cu. ft. per second, which is certainly a remarkably close agreement, under the circumstances.

In reference to the diagram, Fig. 3, attention may be called to the act, known to the speaker from his own experience, that the minimum rainfall and stream-flow of that country for a period extending over many years appears to have occurred in 1895, the year with which the diagram begins. There was not much rainfall in the subsequent winter months, and consequently the flow in the following summer was just about as low as it was the year before. Then came a high spring rainfall, and a greater summer flow. During 1898, there was, in the early months, March, April and May, a very low rainfall, and the diagram ends at the most interesting point, namely, at the end of June, when the line still goes down, and apparently indicates that the minimum flow of the previous years may be exceeded during the fall of 1898. It is to be hoped therefore that the author, in concluding the discussion, will give the Society the benefit of the observations for the following months of July, August and September, 1898, so that a complete report of a very interesting period may be published.

It often happens that formulas are compared with actual observations which are insufficient in number and extent, and bold conclusions are drawn which are not warranted. In this case the author describes a mountainous stream, with an average depth of 2½ to 3 ft., in the bed of which are boulders ranging from 2 ft. to 6 ins. in diameter. These boulders form eddies and cause retardations which must seriously influence the effective cross-sectional area of discharge. Yet with such

conditions the author seriously applies a formula, strictly applicable only to more regular channels; and, because it does not coincide with the actual measurement, makes it appear as though the formula was not as correct as it should be. In the speaker's opinion, all the usual hydraulic formulas, from their very nature, can only be tentatively applied in such cases, because they are deduced from observations made in channels where the conditions are much more regular than those of the stream under discussion. The discrepancies which the author points out are, therefore, only to be expected. They would be reduced only by a much greater number of observations made over the particular cross-section, and by a careful consideration of the effects of every boulder or other obstruction within a reasonable distance, both above and below the section. Mr. Hering.

CORRESPONDENCE.

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E.—The apparent discrepancy between the results of gauging by the use of floats and by meter measurement can be quite easily explained. The value adopted for n was entirely too low for a stream of this class. In the case of torrential streams having a rough channel-bed composed of boulders 2 ft. and less in diameter, it is generally conceded to be unwise to adopt a velocity coefficient, for the formula $v = c\sqrt{r\,s}$, higher than 32, and in most cases where the flow is over ordinary-sized cobblestone beds, the coefficient c will not exceed 53 to 54. These values of c were found to correspond to Kutter's subcoefficient $n = 0.049$ and 0.036, respectively. The author's results would seem, therefore, to be entirely consistent with past experience in gauging such streams, namely, that as a rule the discharge is actually only about half that found by using the ordinary formulas and coefficients. Of course, meter observations are more accurate than, and superior in every way to, floats for obtaining any given discharge quantity; but, on the other hand, the floats are much more readily applied in practice, and, if good judgment is exercised, will generally give very fair results.

The method adopted by the author in gauging the flow of the rivers by simply taking gauge heights and ascertaining the corresponding discharges graphically from a previously constructed discharge curve is all very simple and makes a very convenient record; but there is plenty of room for wide discrepancies unless separate gauge heights are kept, one set for a rising river and another for a falling river, for the well-known reason that for any given gauge-height the actual discharges on a rising river are greatly in excess of the corresponding discharges on a falling river with the same gauge height. Hence, unless all the circumstances of the case are fully known and understood, the simple gauge heights have no great value in determining discharges of rivers.

The rainfall and run-off diagram, Fig. 4, page 11, is certainly quite extraordinary, the annual rainfall taken from the diagram footing up to 103.3 ins., while the annual run-off = 100 ins., or 97% of the rainfall, thus leaving practically nothing to be credited to evaporation, absorption and percolation, which generally get away with 50 per cent. The minimum daily run-off, 138 cu. ft. per second from 143.5 sq. miles of water-shed, means nearly 1 cu. ft. per second from each square mile, which, all things considered, seems to be quite anomalous.

Engineering history shows that rainfall and run-off records constitute a most valuable and reliable guide for the judgment of the hydraulic engineer. Local information peculiar to each district is, of course, especially desirable.

Table No. 3, page 10, calls to mind the fact that grave mistakes are

too often made in storage schemes based on average annual rainfall and run-off. Such calculations are nearly always misleading and fallacious, because the promotors almost invariably fail to recognize the important fact that it is utterly impracticable to catch and hold the excessive run-offs, which must run to waste, and these are the very ones which go to swell the record of the average annual rainfall upon which all the storage calculations are based. Mr. Le Conte.

GEORGE W. RAFTER, M. Am. Soc. C. E.—The paper raises a number of questions of considerable interest to every engineer having to do with the run-off of streams, which the writer wishes to discuss briefly. Mr. Rafter.

In the first place it is desired to emphasize the fact that a current meter gauging does not admit of such accuracy as to justify stating the mean monthly run-offs to even the first decimal place. The lack of judgment which engineers frequently display in the use of decimals to the third or fourth place, when the first place is ample, goes far to substantiate the view held by many people that engineers are not business men; and the use of the first decimal place in uncertain cases, where the even quantity is ample, is equally bad. The working engineer is, first of all, a man of affairs, and should not attempt to appear extremely accurate by the use of decimals, in cases not admitting of decimal accuracy.

In this paper, independent of current meter gaugings, the uncertainty existing as to the relation between run-off, precipitation, and catchment area, easily indicates large room for error. At the same time the writer does not wish to be understood as criticising the paper, either unkindly or captiously. It is rather his desire to take the paper as a text for presenting certain broad propositions in relation to stream flow and allied subjects.

It is becoming quite popular to say that evaporation is very difficult to determine—the average engineer drops the whole subject at this point—and, while the essential truth of the statement cannot be gainsaid, it is equally certain that a good deal can be done with evaporation data, if due regard is given to all that is known on the subject.

As to the view expressed by the author, that it is not likely that there is much loss by evaporation from the Cedar River catchment area, because "during most of the year the air in the mountains is near the point of saturation and the sky is overcast," it may be simply pointed out that if this statement is true for any considerable period of time, then the Cedar River catchment area is apparently an exception to about all other areas thus far studied. Undoubtedly there are, in some cases, periods of greater or less extent during which evaporation is nearly *nil*, but for the entire year the account when properly balanced will always show something to the credit of evaporation. These statements are based upon an examination by the writer of a very large proportion of all the records of run-off in relation to precipita-

Mr. Rafter. tion and catchment area thus far made. It is believed that everything published in the United States has been examined, as well as many of the foreign records.

Systematic study of the English meteorological records extending over many years has shown that in the western part of the British Isles there is usually an increase of precipitation in some degree proportionate to the elevation above sea level. For that particular region the proposition is well established that precipitation increases about in proportion to the altitude above the sea. The "common belief" that this is so, referred to by the author, is probably founded on the general use in this country, up to a few years ago, of English references to run-off and precipitation data. To broadly deny such relation, as the author does in his statement that the Puget Sound record "is a striking illustration of the fallacy of such a supposition," would seem to indicate unfamiliarity with the literature on the subject.

In the State of New York both conditions obtain. The Hudson River catchment area shows a higher precipitation at the mouth of the river than at its source in the Adirondack Mountains, while the Genesee River shows the opposite, namely, higher precipitation at its source than at its mouth. The writer will not consume the space necessary for pointing out why this is so, but will simply refer to his published reports on the Hudson and Genesee storage projects, where very extensive tabulations of the meteorology of these two catchment areas may be found.

The author's statement that "the question of precipitation is not generally of much value to the practicing engineer, etc.," must also be considered somewhat *outre*. No engineer can afford to overlook the three elements needed before questions of stream flow can be discussed properly, *i. e.*, run-off and precipitation for a series of years, and area of the catchment basin. The author is right in assuming that run-off is the most important of the three, but wrong in making excuses for introducing precipitation data.

The author, having studied rather unusual conditions, is disposed to say that any attempt to establish the relation between run-off and precipitation must be futile; and here, again, the writer thinks he has hardly done himself justice. If such a statement means that no broad general formula, covering all classes of streams, is ever likely to be found, then he is undoubtedly on sound ground. If, on the other hand, he means to assert that no formula can be made to fairly fit a given catchment area, then it is equally clear that he has not familiarized himself with what has already been done in that direction.

As a matter of fact, precipitation has its definite laws as well as everything else in the universe, and engineers interested in stream flow questions ought to understand the general trend of such laws. The writer, however, does not wish to be understood as saying that all the

laws governing rainfall apply with equal force to widely separated localities. That some of them do so apply, may be gathered by perusal of a paper, "On Mean or Average Annual Rainfall, and the fluctuations to which it is subject,"* by Alexander R. Binnie, M. Inst. C. E. In this paper Mr. Binnie shows, by an analysis of a large number of precipitation records from different parts of the world, that there are certain fundamental laws applying to precipitation which are about as well established as most of the laws of physics. Mr. Rafter.

Mr. Binnie's paper may be referred to for illustrations of certain well-established laws of precipitation applying to all parts of the world. As an example of fairly well worked-out laws of precipitation, which are not considered as applying to other than a specific drainage area, reference may be made to the writer's report on the Hudson River storage.† Without being exhaustive, it is believed that these two references will furnish enough data to establish the fact that the present state of knowledge of the laws of precipitation is not chaotic.

BENEZETTE WILLIAMS, Esq.—While to those interested in hydraulic gaugings and hydraulic formulas there is much in the paper deserving of thorough discussion, yet the reference to the writer's estimates is scarcely of such nature. It may be well, however, to correct any conclusion which the readers of the paper may form, to the effect that the writer made a gauging of Cedar River. This he did not do. What he did was to estimate the probable discharge of that river, based upon surveys of the slope, and cross-sections, and the observed height of water at three different stages, made by the city engineers of Seattle. The maximum and minimum discharge, stated in the paper as originating with the writer, were evidently taken from his printed "Report on Water-Works for the City of Seattle, to the Mayor and City Council," February 28th, 1890. Mr. Williams.

The following extract from the report itself will convey the best idea of what the writer did with reference to determining the flow of Cedar River:

"There has been no opportunity for a careful hydrographic survey of the river, but such as came incidentally in the course of the preliminary work.

"From the cross-sections of the river-bed, the slope of the stream, and the height of water at different stages of the river, I estimate that on December 21st, 1889, before the snows began to melt, and when the river may be considered as being not much above its low-water stage, the flow was 115 000 000 galls. in twenty-four hours. During February, while the snows were melting, one stage showed 675 000 000 galls., and some days later, at still another stage of the river, probably near the high-water stage, the estimate gives 1 500 000 000 galls. in twenty-four hours. These estimates are rough, and should only be used to indicate the fact that the amount of water that can be furnished by the river, used in connection with Swan Lake as a storage reservoir, will be ample for all possible demands that the city can make upon it."

* *Min. Proc. Inst. C. E.*, Vol. cix. p. 89.
† "Annual Report of the State Engineer and Surveyor of the State of New York, 1896."

Mr. Williams.

From the above it is seen that the minimum of 115 000 000 galls. in twenty-four hours, or 177.9 cu. ft. per second, was the estimated flow of the river at a stage which was thought to be "not much above its low-water stage." The author erroneously states that the minimum discharge, as estimated by the writer, was "not less than 115 000 000 galls. in twenty-four hours." His statement that the writer estimated the maximum discharge to be "about 1 500 000 000 galls. in twenty-four hours," is, perhaps, not so much astray, though, as a matter of fact it is quite different from the language of the report, which says that at a certain stage of the water the estimate was 1 500 000 000 galls. in twenty-four hours, and at this time the river was "probably near the high-water stage." Such language, qualified by the subsequent warning that the "estimates are rough, and should only be used to indicate the fact that the amount of water that can be furnished by the river, taken in connection with Swan Lake as a storage reservoir, will be ample for all possible demands that the city can make upon it," hardly warrants the use of the figures as a determination of the maximum flow. The facts are that the extreme high-water flow of the river had no place in determining the questions in hand, and the assumption that at the particular stage mentioned the river was near high water was based upon a general and unverified statement by the surveyors who did the work. All the engineering work upon which the report was based was done between the latter part of October, 1889, and the latter part of February, 1890, or in less than four months.

To search out a pipe line running 30 miles into a Puget Sound forest, and to make other necessary explorations at that season of the year (when more than half the days are given over to rain or snow) with the force at command, did not permit of much useless work.

The estimates of flow given were based entirely upon the functions of slope and cross-section, with an assumed value for n in Kutter's formula; the slope being assumed constant for all stages of the river. The original data or figures not being at hand, the value used for n cannot now be given.

It may be interesting to compare some of the figures thus obtained with the author's more elaborate determination extending over a period of three years. He found the minimum natural flow, based upon current-meter measurements, as the writer understands it, to be 138.3 cu. ft. per second. The writer estimated that on December 21st, 1889, when the river was considered as being not much above its low-water stage, the flow was 177.9 cu. ft. per second, a difference of a little more than 28 per cent. The author's measurements made by floats were from 40 to 48% greater than his meter measurements when the river was near its low stage. To all appearances this would bring the low-stage determination by the method used by the writer between the float and meter measurements made by the author.

Mr. Williams.

Other determinations by the author are of interest as bearing upon the value of elaborate and involved formulas, such as Kutter's. His statement that "the error due to assuming 0.035 as the value of n is 48% for low stages of the river, while, at the highest stage measured, the error was only 15%, is not reassuring as to the value of such formulas.

The tabulation on page 3 gives a range in the value of n from 0.026 to 0.045. The question arises, of what use is elaboration in a hydraulic formula if such is the case? Why is it not just as scientific to guess at c in the formula $v = c\sqrt{rs}$, as to guess at n in Kutter's involved equation, and also much simpler? It is safe to assert that if it were not for the disposition of so many minds to seek the truth in a maze, the simple form of $v = c\sqrt{rs}$ would never have been departed from, and the painstaking ingenuity that has produced so many fantastic shapes of hydraulic formulas would have been directed to determining the value of c under the varying conditions of practice, much to the advancement of the empirical science of hydraulics.

Mr. Noble.

T. A. Noble, M. Am. Soc. C. E.—The author wishes to express his satisfaction with the interest shown in this discussion by those whose extended experience makes a valuable addition to the paper. It was very far from his intention to leave any impression that might cast reflection on the estimates made by Mr. Williams in his report to the City of Seattle.

In view of the data that were obtained at no small trouble and expense, covering a period of three years and including four dry seasons, it is rather remarkable that Mr. Williams should have guessed so well.

It was supposed, at the time the paper was written, that during the summers of 1895 and 1896, the river had possibly reached the lowest stage it ever would, though the comparatively small discharge during the spring of 1898 would indicate, as Mr. Hering has suggested, a still lower stage during the following summer. The experience of 1898 emphasizes with considerable force the statement in the paper that, "No record of the discharge of a stream is of much value unless it covers a series of years." The lowest point reached at Clifford Bridge, September 19th, 1898, was 72 cu. ft. per second, as against 138 cu. ft. per second on October 31st, 1895. This is such a radical difference that it becomes a matter of vital importance. The year 1898 seems to have been one of unusually low water along the whole Pacific Coast.

Though the complete record of the flow at Clifford Bridge during this summer is not available, yet the flow from Cedar Lake, the record of which is complete since July 1st, indicates that the average flow during the dry season was not much less than in 1896 on account of rains in October as shown in Fig. 5.

Gaugings and a record of the gauge height near the outlet of Cedar Lake, about 15 miles above Clifford Bridge, on Cedar River, have been kept since July 1st, 1898.

Mr. Noble. The gauge height at Clifford Bridge was kept for some timeafter July 1st, in order that, if possible, a comparison could be made with the discharge at Clifford Bridge during previous years. The two curves of low water seem to follow each other in a general way, but without much regularity. The drainage area at Cedar Lake is approximately 80 square miles, and at Clifford Bridge 143 square miles. The relative flow at the two places, from July 1st to September 19th, is shown in Table No. 5.

TABLE No. 5.

Date.	Gaugings at Cedar Lake.		Corresponding Discharge at Clifford Bridge.	
	Cubic feet per second.	Cubic feet per second per square mile.	Cubic feet per second.	Cubic feet per second per square mile.
July 1st.....	357	4.46	510	3.56
October 1st.....	243	3.04	....	....
July 25th.....	110	1.83	207	1.45
September 13th...	46	0.58	84	0.59

Comparative Discharge at the Two Places at Different Stages for the Same Area.

	July 1st.	July 25th.	September 13th.
Clifford Bridge.....	1.00	1.00	1.00
Cedar Lake.....	1.43	1.26	0.98

It should be explained that during June, July, August and September there was practically no rain, and the weather was warm over the entire drainage area. It will be noticed that the discharge gradually decreases until about the first of August, after which time the snow in the mountains being practically gone, the discharge comes almost wholly from springs and seepage. As the snow accumulates mostly in the upper portion of the area, there is a greater proportional discharge in the upper portion of the valley during the spring and the early part of the summer until the snow is gone, when the discharge for the same area becomes practically the same. There are times, however, when it is raining in the lower portion of the area and snowing in the upper portion, and then the discharge per square mile in the lower portion is likely to be greater than that in the upper portion. This ratio of discharge in different portions of the area is constantly changing, being influenced by the amount and distribution of

the rain and snow and by the temperature. At no time is the combination of these influences exactly the same, except in midsummer, when the discharge comes entirely from springs and seepage. Mr. Noble.

At times, when there is an unusual amount of snow on the ground there comes a warm wind and rain, when the run-off becomes enormous, producing floods of a magnitude unknown in any other part of the United States on an equal area.

Ignorance of these conditions has led to mistakes in practically every railroad location in the Sound country, and is now giving serious trouble to an engineer who is putting in a water-power plant in this vicinity and has located the tail race too low.

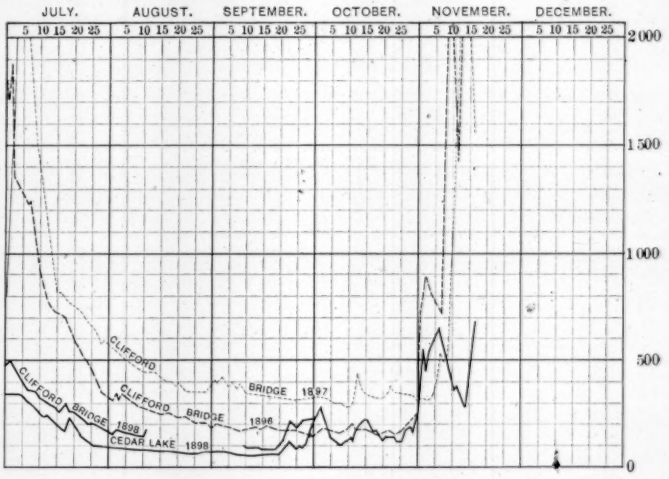

FIG. 5.

The author had no intention of drawing any general conclusions as to the application of Kutter's formula from the particular conditions prevailing in these discharge measurements any more than to point out the danger of using any formula for calculating the discharge in streams of this class. It is not always feasible or possible to take meter measurements, but it is nearly always possible to take submerged float measurements, which, if care is taken in selecting a locality where the cross-section is comparatively even for a short distance, the current slow, and the floats submerged close to the bottom, a fairly accurate measurement can be obtained. The difficulty in obtaining accurate data for calculation by Kutter's formula is often as great and sometimes greater than it is to take a submerged float measurement, and it has the further disadvantage of leaving the element

Mr. Noble. of the character of the stream to be judged by the engineer. This may also be said regarding the estimation of run-off from precipitation. It is generally easier, and far more satisfactory, to take measurements from the stream itself, particularly in a broken and uneven drainage basin. It would be necessary in a rough country to keep records at an almost infinite number of places, to determine the average rainfall, since it changes very radically in comparatively short distances. When a record of precipitation has been kept for some time in a certain drainage area, it becomes of considerable value to compare it with discharge measurements taken simultaneously at the same point.

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TRANSACTIONS.

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No. 843.

STAVE PIPE—ITS ECONOMIC DESIGN AND THE ECONOMY OF ITS USE.

BY ARTHUR L. ADAMS, M. Am. Soc. C. E.

PRESENTED OCTOBER 19TH, 1898.

WITH DISCUSSION.

This subject, in one or more of its aspects, has been briefly dwelt upon at various times by engineers when describing the construction of hydraulic works involving the use of pressure pipe of this type, but a full discussion of the various practical and economic problems involved in stave pipe design and the determination of its real place among standard types of pressure pipe have yet to appear.

The great dissimilarity in practice indicates either a very general unfamiliarity with the principles involved, or much difference of opinion regarding their practical application. That there is need for a better understanding of these principles, and of the correct limitations of the pipe's successful and economic usefulness, is apparent from the not infrequent partial disasters that have attended its attempted construction. That it has to-day a place among standard types of construction is apparent from the remarkable degree of favor accorded it by everyone who has used it intelligently. The author firmly believes that the use of this pipe, now chiefly confined to the West, is destined

to receive greatly extended recognition among engineers everywhere. These considerations seem sufficient to warrant this presentation, which it is hoped may serve as a starting point for the evolution of clearer and more generally accepted ideas relative, not alone to the details of stave pipe design, but to the broader question of the comparative economies of different classes of pressure pipes, a question which recent important experiments and accumulating experience have done much to reopen to argument.

In treating the subject, the author proposes to discuss the essential considerations in stave pipe design, and some of the practical and theoretical limitations imposed. Supplemental thereto a study will be made of several existing pipe lines, with reference to the details and factors used and the attendant results, and, finally, having outlined a simple and rational method for making a safe and economic design, a comparison will be made between the leading types of large pressure pipes, with reference to their relative first cost, durability and carrying capacity, from which the author will seek to determine the rightful place of stave pipe in economic construction.

General Type Considered.—At first stave pipe was principally used for penstocks in the development of water power, and was built in tapered sections which were put together after the manner of stove pipe. Apparently the first instance of its use as a continuous tube was the 6-ft. penstock constructed at Manchester, N. H., in 1874, by J. T. Fanning,* M. Am. Soc. C. E., and which is still doing service. Round bands were suggested by certain letters patent as early as 1880, and were first put into extensive use at Denver, Colo., in 1883. The particular type referred to in this paper is that which has been thus evolved, *i. e.*, a pipe which is built continuously in the trench, of staves of variable length, having radial edges and concentric faces, and which are held together by metal bands usually circular in section and spaced in accordance with the strains imposed.

Essential Considerations.—In a former contribution by the author† the following appears:

"In the design of a wood-stave pipe, the following essential points require consideration: The staves must be thin enough to secure complete saturation and to deflect readily to the degree of curvature employed, and they must be thick enough to prevent undesirable

* *Transactions*, Am. Soc. C. E., Vol. vi, p. 69.
† *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 15.

percolation through them. The bands must be of such size that, when spaced to secure the desired factor of safety against rupture, there will at the same time be no sensible flexure in the staves and no destructive crushing of the fiber beneath the bands. While fulfilling these conditions, the proportion between the thickness of the staves and the strength and spacing of the bands must be such that the swelling of the wood will not produce injurious strains upon what might otherwise be a properly proportioned band."

These are the primary considerations, generally stated, which should govern all design, and the author believes it will be profitable to consider them more in detail, to point out some of the limitations of each, the effect of certain practical considerations, and to reduce the whole to a form easy of application. To do this intelligently a clear conception of the nature and source of the probable strains set up in the construction and operation of the pipe should first be secured.

Sources of Strains.—The tensile strains resisted by the bands are from three sources:

(1) The initial strain, caused by cinching during construction.

(2) The pressure of the water within the pipe.

(3) The swelling of the staves.

The strains resisted by the staves are from the following sources:

(1) The compressive strain of the bands.

(2) The compressive strain upon the edges of adjoining staves.

(3) The pressure of the water producing flexure of the staves between adjacent bands.

The fact that most of these strains, in their amount and importance, are more or less inter-dependent makes any separate determination of the values of each impossible without coincidently determining the nature of their mutual relation under some variety of conditions.

Cinching Strain.—It is evident that to secure a pipe that shall be tight, or practically so, when first filled, the initial cinching strain must be sufficient in extent to produce:

(1) A degree of compression per square inch between the staves in excess of the water pressure.

(2) A uniform contact between the band and all the staves; and, if the band be of a curved section, a sufficient indentation of the wood to afford the band a bearing contact sufficient in area to resist, without further immediate indentation, the water pressure incident to the filling of the pipe.

If these conditions are not complied with, it is evident that leakage must ensue until the necessary compression between the staves and the requisite area of contact between the bands and the staves are supplied by the swelling of the staves. Whether this will be accomplished or not will depend chiefly upon the degree of seasoning of the staves. That the question of seasoning, in securing tight pipe, so long as the staves are uniform, is relatively unimportant, with proper cinching, seems to follow logically, though the practical difficulties incident to the handling and shipping of the heavier green timber, and the prevention of unequal shrinkage before getting the staves into the pipe, usually make the at least partially seasoned timber the cheaper. It is apparent that back-cinching, to a degree producing greater compressive strain than is essential to the foregoing requirements, is to that extent a needless addition to the cost of erection and to the band strain.

Owing to the friction between the band and the staves, and the compressive resistance of the wood, it is not found possible in practice to secure the necessary area of contact, under a band of curved section, by screwing up the nut at the end of the band, without inducing an objectionably high strain. To accomplish the result in a better way, the band is usually pounded vigorously at all points, simultaneously with the use of the wrench. Such a practice is highly commendable, and is, indeed, usually essential, as it tends to secure the proper degree of indentation with the least residual tensile strain in the band consistent with securing the requisite compression between the staves.

Just what this cinching strain usually amounts to in practice depends upon the character of and manner of doing the work, and is, probably, as variable as individual judgment. So far as it influences the success of the work, it is believed that the foregoing sufficiently elucidates the principles involved. In its effect upon the final working strain in the band, it will be shown later that a considerable variation in practice within reasonable limits will not seriously affect the final stresses, though, doubtless, much affecting the temporary strains resulting on first filling.

Water Pressure Strain.—The strains induced in the bands by the pressure of the water within the pipe are susceptible of positive determination, and the methods of their solution are too well known to require any explanation herein.

Strains from Swelling Staves.—The additional strain caused by the swelling of the staves is less easily determined, and exists only within certain limitations of the first afore-mentioned source of strain.

With swelling timber, as with a steel spring, it may be assumed that its expanding power, at a given stage of compression, well within what may be called its elastic limit, is measured by the load necessary to produce that compression under like conditions. Hence its ultimate swelling power, at a given degree of compression, is the measure of its ultimate resisting power. It is also influenced by the hardness, direction of grain and such other conditions as affect the compressive strength of timber within its elastic limit when resisting forces applied across the grain.

If, then, the staves in a pipe be compressed laterally by the bands beyond the elastic limit of the saturated wood, there can be no swelling of the staves, and, consequently, no increased strain in the bands from this source. On the contrary, a farther compression of the fiber will result until an equilibrium is established between the compressive strain and the sustaining power of the wood; and if the sustaining power is below that requisite to resist the pressure of the water, leakage will, of course, result. If, on the other hand, the compression is less than the corresponding swelling power of the wood at the degree of seasoning and compression used, the timber will, by swelling when wet, develop an expanding power in some degree commensurate with the ratio of the volume of the uncompressed saturated stave to the same when compressed to the degree used, which expansion, of course, induces a correspondingly increased strain in the bands.

Thus there results the rather remarkable condition that if the bands, in resisting the water pressure, tax the staves beyond the limit of their permanent compressive strength, they, by yielding, release the excess of pressure, and leakage results, and if less is required of them than the most they are capable of giving, they force the band to take all the additional strain they are capable of exerting.

The amount of additional band strain thus induced is, doubtless, nearly proportional to the area of contact between the staves in the straight seams. The avoidance of unnecessary thickness of staves, too long band spacing and the providing of such an area of contact between band and staves as shall give a necessary working compressive strain on the wood, not in excess of its permanent resisting strength,

and only sufficiently below it to make certain of its not being exceeded by the water pressure, ought, therefore, to commend themselves as prime essentials in avoiding unnecessary and perhaps injurious band strains.

The special advantage of a band of curved section, as contrasted with a flat band, now becomes apparent, since by the former a permanent compressive strain on the wood is provided sufficient to produce a decided indentation. The swelling of the staves, if any, and the accompanying increased compressive strains beneath the band are, therefore, followed by a farther yielding of the wood; and as the band strain increases, the area of contact with the band is increased by the farther indentation of the stave. With flat bands the unit compressive strain beneath them would almost invariably be so low that no indentation of the stave would result, and swelling strains to the full exerting power of the wood would be added to the band strains arising from other sources.

Temporary and Permanent Strains.—From the foregoing it is apparent that the strains resulting on first filling a pipe, or temporary strains, the necessary strains for permanently maintaining a tight pipe under pressure, and the permanent strains finally obtaining in the pipe, are three separate and usually very different quantities.

The permanent strains in a band are evidently the sum of those induced by final cinching, by water pressure and by the transverse swelling of the staves, subject only to the limitations imposed by the sustaining and swelling power of the wood when saturated.

The band strain necessary to maintain a tight pipe has already been shown to be that requisite to resist the water pressure and to maintain any excess of pressure between the staves needed to prevent seam leaks.

The temporary strain induced on first filling is the sum of the final cinching strain, the water pressure strain, and that arising from a swelling of the staves, subject only to the limitation of the immediate compressive resistance of the wood under the band while still dry, a limit scarcely reached in any proper construction.

Strains Resisted by Staves.—In discussing the causes and effects of band strains, the character of the strains resisted by the staves have been made sufficiently plain. Their further influence upon proper design will be considered later.

Equations Deduced.—In the following equations for the various strains previously considered, the symbols used are as follows:

R = the internal radius of the pipe,
r = the radius of the band section,
t = the thickness of the stave, in inches,
f = the spacing of the bands between centers, in inches,
S = the tensile strain in the band, in pounds,
s = the safe tensile strain in the band, in pounds,
E = the permanent sustaining power of the staves, in pounds per lineal inch of band,
E' = the temporary sustaining power of the staves, in pounds per lineal inch of band,
E'' = the permanent swelling force of the wood, in pounds per square inch,
E''' = the temporary swelling power of the wood, in pounds per square inch,
P = the water pressure, in pounds per square inch,
e = the safe bearing power of the wood, in pounds per lineal inch of band,
X = the additional strain induced by band cinching, in pounds.

The least band strain induced by water pressure $= P\,R\,f$.

The permanent band strain induced by swelling of staves $= f\,t\,E''$.

The least permanent strain in the bands of a tight pipe $= (X + P\,R\,f + f\,t\,E'') < R\,E$. Assuming $X = \frac{3}{2}\,t\,f\,P$ and substituting, we have as the least permanent value of S for a tight pipe when $P < E''$

$$S = \left(P\,f\,(R + \tfrac{3}{2}\,t) + E''\,t\,f\right) < (R + t)\,E \ldots\ldots\ldots (1)$$

If $P > E''$, this becomes

$$S = \left((R + \tfrac{3}{2}\,t)\,P\,f\right) < (R + t)\,E \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

which is the equation generally used for computing band strains for all pressures.

The least temporary band strain induced on first filling a tight pipe $= (X + P\,R\,f + E'''\,t\,f) < (R + t)\,E'$. Assuming, as before, $X = \frac{3}{2}\,t\,f\,P$ and substituting, we have as the temporary value of S when $P < E'''$

$$S = \left(P\,f\,(R + \tfrac{3}{2}\,t) + E'''\,t\,f\right) < (R + t)\,E' \ldots\ldots\ldots (3)$$

If $P > E'''$, the equation becomes the same as (2).

From the foregoing the following equations, convenient for application, are deduced:

When $P < E''$:

$$f = \frac{s}{(R + \frac{3}{2}t)\ P + E''\ t}, \text{ when } s < (R + t)\ e \ldots\ldots (A)$$

$$f = \frac{(R + t)\ e}{(R + \frac{3}{2}t)\ P + E''\ t}, \text{ when } s > (R + t)\ e \ldots\ldots (B)$$

When $P > E''$

$$f = \frac{s}{(R + \frac{3}{2}t)\ P}, \text{ when } s < (R + t)\ e \ldots\ldots (C)$$

$$f = \frac{(R + t)\ e}{(R + \frac{3}{2}t)\ P} \text{ when } s > (R + t)\ e \ldots\ldots (D)$$

Inasmuch as stave pipe has never been used under heads where the value of P is greater than E'', and since, as will subsequently be seen, it has not as great economic value under such heavy pressures as under lighter pressures, formulas (C) and (D) have no great practical value.

Sustaining and Swelling Power of Staves.—In order to make use of the formulas, it is essential that proper values be determined for E, E', E'', E''' and e.

Since the sustaining power of wood depends upon the degree of its permissible compression within the limit of its ultimate crushing, it is necessary to assume some fixed amount of stave indentation as a basis for determining the probable values of E and E'. Now, in the actual process of pipe construction, it has been frequently observed that an indentation equal to one-eighth of the radius of a round band, of such sizes as other considerations render suitable for use, may be produced in the stave without breaking the fiber, while any considerable increase beyond this point has a destructive effect. This degree of indentation gives a bearing surface equal to half the diameter of the band, which may, therefore, be assumed as the greatest that can be safely obtained.

In considering contact between cylindrical bodies and the flat surfaces of saturated timber, the ordinary experimental data relating to flat surfaces having uniform contact with seasoned timber are scarcely

applicable. The scarcity of experiments influencing the correct determination of these values is to be regretted, and it is hoped will yet be supplied. So far as the author is aware, the only experiments available are those made by D. C. Henny,* M. Am. Soc. C. E. A further study of these experiments forecasts some interesting conclusions regarding these values, and additional light is derived from a study of the strains prevailing in some existing pipe lines, as will be shown later. The experiments mentioned were made to determine the swelling power of Oregon "yellow fir." For convenience, the results are here given:

TABLE No. 1.

Date of Immersion.	Date of observation.	Observed strain in bands, in pounds.	Corresponding strain, per square inch between staves.	Corresponding strain on staves under band, per lineal inch of band.	Corresponding strain on staves under band, per square inch of contact.	Remarks.
			Section 1, No. 1.			
Feb. 27	Feb. 27	4 750	288	456	1 830	Initial strain before submerging.
	Mch. 1	3 400	206	328	1 310	
	" 4	2 950	179	284	1 140	
	" 7	2 750	167	265	1 060	
	" 9	2 525	153	243	970	
	" 12	2 525	153	243	970	
			Section 1, No. 2.			
	" 12	100	7	10	38	Band was loosened to the strain indicated.
	" 18	1 500	91	145	580	
	" 21	1 500	91	145	580	
	" 26	1 550	94	150	600	
			Section 1, No. 3.			
	" 26	500	30	48	190	Band again loosened to the strain indicated.
	April 1	1 000	61	97	390	
	" 4	1 050	64	101	400	
	" 8	1 100	67	106	420	
	" 11	1 200	73	116	460	
	" 15	1 225	74	118	470	
			Section 2, No. 1.			
Apr. 16	" 16	450	33	43	170	Band was first drawn to tension of 5 000 lbs. and then released to the strain indicated.
	" 17	1 400	85	135	540	
	" 20	1 900	115	183	730	
	" 24	2 000	121	192	770	
	" 27	1 975	120	190	760	
	May 1	1 900	115	183	730	
	" 4	1 900	115	183	730	
	" 7	1 800	109	173	690	
	" 10	1 700	103	164	660	
	" 13	1 600	97	154	620	
	" 17	1 600	97	154	620	

In these experiments the bands were depressed into the staves until a bearing of half their diameter was secured.

* "The Astoria City Water-Works." *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 18.

In experiment No. 1, with Section 1, it will be noted that a very heavy initial tension was put upon the band, which the staves after being immersed could no longer withstand. The resulting compressive strain resisted by the staves under the bands accordingly fell from 456 lbs. per lineal inch of band to 243 lbs., or 970 lbs. per square inch of contact, at which point it became stationary, although, the staves being at this stage probably incompletely saturated, a longer immersion might have resulted in producing a slightly lower permanent resistance. The result is strongly indicative that this pressure of 970 lbs. per square inch of contact is the utmost that in practice can be permanently resisted by the staves, while the subsequent experiments show that hard cinching and a full band-bearing as here used are necessary to secure nearly so high a resistance. As will be noticed later, the correctness of this general deduction is practically substantiated by a study of some existing structures.

The resulting compression between the staves in this experiment is seen to be 153 lbs. per square inch, a figure from which no conclusion can be drawn other than that the diminution in the band strain must have been the result of yielding under the band rather than between the staves.

Experiments Nos. 2 and 3, with the same section, in which the band tension was successively released by means of the wrench, are evidently well calculated to determine the approximate swelling force exerted laterally by the staves. The staves having been compressed for a period of two weeks, to a degree under the bands well beyond the limit of the wood's elasticity, it is evident that a very slight lengthening of the band by unscrewing the nut would be immediately followed by the partial or entire release of the band strain. The amount of lengthening necessary for this result would doubtless be but a small fraction of the total of, say, 3% expansion by swelling, of which the kiln-dried timber would be capable, if unresisted. The staves, not having been compressed laterally in the previous experiment to a degree in any way calculated to impair their swelling power, were then free to develop this power to the utmost within the limit of their ultimate bearing power under the bands, which the previous experiment has shown to be not less than 970 lbs. per square inch of contact. On the first release of the band strain, No. 2, the staves developed a swelling power of 94 lbs. per square inch. On the second

release, they developed a power of 74 lbs. per square inch, the power diminishing, as might be expected, as the wood gradually approached its full unrestrained proportions.

The experiment with Section 2 is a confirmation of the results derived from experiment 2, on Section 1. The pipe was but slightly cinched after the band had been brought to a proper bearing. The staves developed a swelling power of 97 lbs. per square inch, a figure almost identical with that given by the previous experiment, while the resulting compressive strain under the band, 620 lbs. per square inch, was not such, under the prevailing conditions, as to be likely to cause a degree of yield which would vitiate the result.

So far as they go, these experiments demonstrate: (1) That with a proper degree of compression, an ultimate permanent pressure of between 900 and 1 000 lbs. per square inch of contact with the band may be resisted by the staves. This result may probably be conservatively assumed to obtain in the use of all bands of prevailing sizes, and of staves without regard to the character of the grain. (2) That the ultimate swelling power of the wood may be taken at from 90 lbs. to 100 lbs. per square inch of stave contact in straight seams, a result which may be effected by the spacing of the bands, since a slight flexure in the stave would tend to release the strain instead of transmitting it to the bands. (3) That for water pressures exceeding, say, 94 lbs. per square inch, and probably considerably less, hard cinching, rather than the swelling of the wood, must be relied upon to maintain a tight pipe. (4) That the yielding quality of the wood renders hard cinching not seriously objectionable in its effect upon final band strains with the usual factors of safety in the band and when working reasonably close to the limit of ultimate compressive resistance of the stave.

As it has been observed in practice that the difference in the compressive strength of Douglas fir and redwood is not very great, when both are saturated, for present purposes no distinction will be made between them.

One of the most important points to be determined from the foregoing is the maximum value for e which can be used with entire safety in designing a pressure pipe. It will be shown later that pressures as high as 1 100 lbs. per square inch have been successfully used under favorable conditions, but the logical conclusions drawn from the pre-

vious discussion, as well as some results from actual experience given later, strongly indicate that such an assumption is insufficiently conservative for entirely safe construction. With thorough compression of the bands into the staves, well-seasoned lumber, skilled construction and entire immunity from water-hammer, it would seem that this value should not be taken much, if any, above 800 lbs. per square inch, while if subject to any considerable fluctuations of pressure, or to doubtful constructive skill, a much lower figure should be used.

In view of the foregoing, the following values are assigned for use in the formulas previously given:

E = 240 lbs. for ½-in. bands, when the fiber is fully compressed (960 lbs. per square inch).

E' = say 500 lbs., in the absence of any experiment.

E'' = 95 lbs. for 12-in. band spacing. Assumed at 100 lbs. for all spacings.

E''' = 120 lbs. for 12-in. spacing. Assumed at 125 lbs. for all spacings.

e = equivalent of 800 lbs. per square inch of band contact or less.

Determination of the Proper Size and Form of Band.—Before making use of any formula for the determination of proper band spacings, it is necessary to determine the size and form of section of the band that will most economically fulfill the requirements of any particular case in hand. The selection will be influenced by the following considerations: Bands less than ⅜ in. in diameter are not suitable for use chiefly because of torsional weakness and difficulty of upsetting. An increase in the permissible value of e may be obtained without increase of metal section of band by using an oval form, which can be obtained at only a slight increase of cost above the rounds of similar weight. The price per pound for the bands is less for the heavier than the light sections. The use of upset rods or rolled threads is always economical, the latter being usually preferable because of the more perfect workmanship. The heavier the bands used, the fewer to be handled, the less the cost for both material and labor, and the better the condition of the band section for resisting corrosion, though the added stiffness, increasing the difficulty of placing, will in the larger sizes partially offset these advantages.

Referring to the formulas for band spacing, it is seen that economy in the use of the steel requires the selection of a band of such sectional

area that if possible $s < (R + t)\, e$, as otherwise it is necessary to use closer spacing—more steel—than the pressure requires, in order to secure a sufficiently low value for e.

By writing $s = (R + t)\, e$, the greatest value of s for any given band section that can be used consistently with a desired value of e, may be determined; and, by substituting for s and e their proper values corresponding to bands of various standard sizes, the limitations of the economic use of each may be ascertained, thus:

$$R + t = \frac{s}{e}.$$

TABLE No. 2.—Minimum Sizes of Pipe to which Specified Bands are Applicable without Exceeding Values of 650 Lbs. and 750 Lbs. for e, and a Factor of 4 for the Bands.

Size of band.	s (= ¼ ultimate strength.)	Band pressure per square inch.	e (Band pressure per lineal inch.)	Least external radius of pipe.	Band pressure per square inch.	e (Band pressure per lineal inch.)	Least external radius of pipe.
⅜ in.....	1 650 lbs.	650 lbs	122 lbs.	13.5 ins.	750 lbs.	140 lbs.	11.8 ins.
½ "	2 250 "	"	142 "	15.8 "	"	164 "	13.7 "
⅝ "	2 950 "	"	163 "	18.1 "	"	187 "	15.7 "
¾ "	3 725 "	"	183 "	20.4 "	"	211 "	17.65"
⅞ "	4 600 "	"	203 "	22.6 "	"	234 "	19.6 "
1 "	6 600 "	"	244 "	27.0 "	"	281 "	23.5 "

A factor greater than 4 must therefore be used in the steel in designing pipes of smaller diameter, if bands of round section are used and the specified values of e are not exceeded. The band strains to be used are readily determined by the same formula as before indicated:

$$s = (R + t)\, e.$$

By the use of a band of oval section, a greater bearing surface is secured between the band and the stave with the same degree of indentation, thus permitting the use of less steel for given values of e, in designing pipes of small diameter. For this reason they have been already introduced and used by the Excelsior Wooden Pipe Company of San Francisco on a number of pipe lines, though not with the economy that would now attend their use, as manufacturers have not until recently been able to quote the present figures, which are but little in excess of the price for rounds. These conditions will doubtless lead to the general use of the oval section on small pipes,

TABLE No. 3.—GREATEST BAND STRAINS CONSISTENT WITH SPECIFIED VALUES OF e FOR PIPES OF SMALL DIAMETER.

$\frac{3}{8}$-in Round Bands.

Internal diameter of pipe.	External radius of pipe	s (= Assumed working strain on band).	Factor of safety in band.	s (= Assumed working strain in band).	Factor of safety in band.
	e = 122 lbs. = 650 lbs. per square inch.			e = 140 lbs. = 750 lbs. per square inch.	
12 ins.....	7.1	866 lbs.	7.6	994 lbs.	6.1
16 "	9.2	1 122 "	5.9	1 288 "	5.1
20 "	11.2	1 366 "	4.8	1 568 "	4.2
24 "	13.5	1 650 "	4.0		

though any departures from the round sections have the disadvantage of exposing a greater relative surface to corrosive action.

The use of an oval section in the above cases would result in a saving of about 17% of the steel.

Band Spacing.—If, in accordance with the foregoing, the proper band is determined for a pipe of given diameter, formulas A, B, C or D may at once be applied to determine the proper spacing for any pressure and thickness of stave, the selection for any case being made by observing the relative values first of P and E, and then of s and $(R + t)\, e$ as indicated.

Requisite Thickness of Staves.—The staves must be thick enough to prevent percolation, to resist undue flexure with the band spacing adopted, to prevent cracking or crushing of quarter-sawed staves during the process of cinching, and to afford the rigidity necessary to support the weight of the pipe and back-filling without distortion. With these requirements met, the staves, for reasons already mentioned, should be as thin as possible.

Concerning percolation through the staves, the author has already pointed out* that in using Douglas fir under pressures in excess of about 80 ft., vertical and bastard-grained staves only should be used if the staves are $1\frac{3}{8}$ ins. in thickness. With this precaution trouble will not arise from the splitting of quarter-sawed staves, and this thickness will be found sufficient to resist percolation at any pressure under

* "The Astoria City Water-Works," *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 16.

which stave pipe is likely to be used. Redwood, being of finer grain, gives no trouble from percolation.

If the size of band for a pipe of any diameter, and the band spacings are determined in accordance with the foregoing principles, staves of the thicknesses generally used will always be safe against undue flexure if a maximum spacing of 12 ins. is adopted. This is by no means an extravagant assumption, since this spacing will in most cases correspond to a very light water pressure; and if, when designing a pipe of large diameter, the band selected gives an unnecessarily large factor under light heads with this spacing, a smaller band can be substituted.

A practical consideration that has influenced very properly the thickness of staves is the desirability of running them from timbers of commercial sizes. Thus, by enabling the mill to dispose of rejected pieces, a lower price on the accepted staves is secured.

The practice of running staves from 2 ins. x 6 ins., 2 ins. x 8 ins. and 3 ins. x 8 ins. stock nearly meets the economic requirements of pipes designed in accordance with the foregoing principles, which requirements will, however, be a little better conserved if $1\frac{1}{2}$ ins x 4 ins. is used for the smallest diameters, and if the odd size $2\frac{1}{2}$ ins. x 8 ins. is used for certain sizes of pipes intermediate between those for which 2 ins. x 6 ins. and 3 ins. x 8 ins. are used.

A careful study of these and other requirements points to the following as being for general use the best balance between the various influences affecting the relation between pipe diameters and the size of stock for staves, the staves in each case being run as thick as the stock will permit.

Pipe diameters.	Stock sizes.
10 ins. to 14 ins.	$1\frac{1}{2}$ ins. x 4 ins.
16 " to 48 "	2 " x 6 "
50 " to 58 "	$2\frac{1}{2}$ " x 8 "
60 " to 72 "	3 " x 8 "

Suggested Standard Designs.—Assigning to e the conservative value of $\frac{650}{r}$, the author ventures to submit the following table of pipe diameters, with the corresponding stave thicknesses and character of bands, which, for the conditions of general use, he believes to be as closely in accord with sound theory and economic practice as the

present state of investigation and the constructive art will permit. The economy of these relative proportions is somewhat influenced by the locality of construction as affecting freight rates on the steel, but not to a serious extent. Should it be desired to use some other value of e than that designated, the needed modification can be made without difficulty.

TABLE No. 4.—Economic Proportions for Pipe Design.

Nominal diameter.	Stock sizes for staves.	Thickness of finished staves.	Economic sizes of bands.	s (Permanent band strain).	Factor of safety in band.	e (Maximum values $= \frac{650}{r}$.)
Inches.				Lbs.		Lbs.
10	1¼ ins. x 4 ins.	1 1/16 ins.	5/16 x 7/16 in.	1 255	5.26	207
12	1¼ " x 4 "	1 1/8 "	5/16 x 7/16 "	1 475	4.47	207
14	1½ " x 4 "	1 7/16 "	5/16 x 7/16 "	1 650	4	.....
16	2 " x 6 "	1 5/32 "	5/16 x 7/16 "	1 650	4	.....
18	2 " x 6 "	1 3/8 "	5/16 x 7/16 "	1 650	4	.....
20	2 " x 6 "	1 3/8 "	5/16 x 7/16 "	1 650	4	.....
22	2 " x 6 "	1 3/8 "	⅜ in.	1 508	4.4	122
24	2 " x 6 "	1 3/8 "	⅜ "	1 650	4	.....
27	2 " x 6 "	1 7/16 "	⅜ "	1 650	4	.....
30	2 " x 6 "	1½ "	½ "	2 673	4.4	162
36	2 " x 6 "	1 7/16 "	½ "	2 950	4	.....
42	2 " x 6 "	1⅝ "	½ "	2 950	4	.....
48	2 " x 6 "	1 11/16 "	½ "	2 950	4	.....
54	2½ " x 8 "	2⅛ "	⅝ "	4 600	4	.....
60	3 " x 8 "	2⅛ "	⅝ "	4 600	4	.....
66	3 " x 8 "	2 7/16 "	⅝ "	6 600	4	.....
72	3 " x 8 "	2⅝ "	¾ "	6 600	4	.....

Some Existing Pipe Lines.—In seeking data concerning pipe lines already constructed, the author has encountered the customary indifference or unwillingness on the part of many engineers to divulge anything pertaining to the methods used in the design and execution of the work done under their supervision. Especially has this been the case where defects of a serious nature have developed. This is, of course, a matter for regret, since the most instructive deductions can usually be made from such experiences. Chief among the works which might well be made instructive to the profession, but which, for the above reasons is necessarily here omitted, is the 36-in. and 38-in. outfall sewer of the City of Los Angeles, Cal.

On the other hand, the author wishes to express his grateful recognition of the courtesies extended by those engineers whose appreciation of the advantage to the profession arising from a general discussion of engineering subjects by engineers has largely made possible the preparation of this paper.

TABLE No. 5.—STRAINS IN SOME EXISTING PIPE LINES.

Number.	Location of pipe.	Date of construction.	Nominal diameters, in inches.	Length, in miles.	Thickness of staves, in inches.	Kind of wood.	Character of round bands.	Ultimate strength of bands.	Maximum band spacing, in inches.	Feet pressure, for maximum spacing.	Minimum band spacing, in inches.	Feet pressure, for minimum spacing.	Band strain, from water pressure.	Band strain, from least practicable cinching.	Resulting band factor, neglecting strain from swelling staves.	Band strain, from swelling staves.	Resulting factor in band, including all strains.	Least band pressure on staves per linear inch, to maintain tight pipe.	Least band pressure on staves per square inch, to maintain tight pipe.	Total probable band pressure on staves, per linear inch of band.	Total probable band pressure on staves, per square inch of band contact.
1	Denver, Colo.	1880	30	18.00	1⅛	.....	1½-in. upset	11 800	12 (?)	32	2.23	172	2 500	538	3.90	2 010	2.34	182	729	208	1 212
	" "												2 500		3.90	373	3.46	182	729		880
2	Provo, Utah	1901	14	3.70	1⅛	Redwood	¾-in. plain.	5 800	18	225	2.75	165	1 967	402	3.00	.....	.....	211	1 126	.....	.....
	" "										6.00	50	3 515	3827	1.64	1 005	1.30	134	1 108	258	1 349
3	Caldwell, Idaho	1900	54	0.13	1⅛	Pine	¾-in. plain.	6 300					1 944	3096	3.00	378	2.50	216	536	169	676
4	Butte, Mont.	1902	24	9.00	1⅛	Redwood	¾-in. upset.	6		82	2.44	202	2 502	461	3.9	864	3.04	225	900	260	1 156
	" "												2 502	461	3.9	351	3.50	225	900	251	1 004
5	Bear Valley, Cal.	1903	52	0.41	2.00	Redwood	¾-in. upset.	17 300	12	228	2.00	165	3 791	4377	4.06	2 400	2.60	151	488	236	757
	" "				2.60								3 724	5584	4.00	5 590	3.58	150	479	168	537
6	Logan, Utah	1908	18	1.60	1⅛	Redwood	¾-in. plain.	5 300	12	18	4.00	53	828	2002	5.08	1 728	1.91	100	538	266	1 417
	" "												828	2000	5.08	5 576	3.30	99	527	154	826
7	Los Angeles, Cal.	1905	24	1.00	1⅛	Redwood	*	9 000	12	25	4.00	88	1 502	969	4.92	1 650	2.58	137	414	220	785
	" "												1 683	3815	4.20	5 550	3.94	161	496	202	610
8	Astoria, Ore.	1905	18	7.50	1⅛	Fir	⅞-in. upset	9 000	12	25	2.95	175	1 166	2960	5.56	1 650	2.75	138	681	297	1 358
	" "												1 588	3632	4.76	309	4.10	182	852	212	970
9	Hollister, Cal.	1906	12	.....	1⅛	Redwood	¾-in. rolled thread	6 600	10	28	2.75	100	599	169	8.50	1 135	3.48	108	575	206	1 417
	" "			.....									716	2011	7.20	309	5.38	129	696	172	918
	" "		14	.....	1⅜	.....	¾-in. rolled thread	.....	11	22	2.75	100	785	182	7.20	1 272	3.01	112	600	208	1 430
	" "			.....		.....		.....			2.75	100	885	207	6.33	318	4.85	128	680	167	969
10	St. Paul, Minn.	1906	42	1.50	1⅛	Fir	¼-in. plain.	7 500	14	5	4.00	25	698	90	10.45	2 450	2.96	132	1128	141	563
	" "												911	114	7.32	700	4.35	46	182	77	306
11	West Los Angeles, Cal.	1906	30	7.50	1⅛	Redwood	¼-in. rolled thread.	11 800	12	20	.....	.....	1 502	276	6.42	1 800	3.25	111	446	220	882
	West Los Angeles, Cal.																				
12	Ogden, Utah	1907	72	5.10	2.00	Fir	⅞-in. upset.	18 400	5.25	55	3.50	105	2 970	3555	4.33	525	3.63	165	660	197	788
	" "										4.00	100	4 490	3974	3.78	1 050	3.11	128	409	156	487
	" "										2.5875	100	4 490	3974	3.78	575	3.38	128	409	143	447
	" "										3.54	117	6 471	5389	3.77	708	3.42	184	402	170	454
13	West Los Angeles, Cal.	1908	14	7.50	1⅛	Redwood	⅞-in. rolled thread.	5 500	10.00	20	.....	.....	605	194	6.88	1 136	2.81	98	524	287	1 264
	West Los Angeles, Cal.																				
										4.50		66	998	217	4.93	506	3.40	137	730	166	885

* Oval bands, ⅞ in. x ¼ in., were used.

From the many existing lines of stave pipe, thirteen have been selected for presentation. These, it will be observed, cover a great range in both size and pressure, and are, with some exceptions, characteristic of the best practice prevailing at the time of their construction. There would be little gained by pointing out defects in design of a nature that would be patent to any intelligent person, for which reason no cognizance is taken of any existing pipes that have been designed or constructed foolishly.

Table No. 5 is believed to be self-explanatory, in the light of the previous discussion. The chief interest centers in the unit band strains used, the probable effect upon these of the swelling of the staves, and the necessary compressive resistance of the staves. Of the latter, it is, of course, the maxima that are chiefly interesting, and these, for reasons already made plain, are usually found in pipes of relatively small diameter, since with large pipes other considerations lead to lower values.

Band Strains.—The factors used in proportioning the spacing of the bands, disregarding the strains induced by swelling timber, vary from 1.64 to 10.45, while the factors resulting from a consideration of the stave swelling range from 1.30 to 4.35. This latter source of strain seems to have received no recognition in design previous to 1895. Since then, in the design of the Los Angeles, Astoria, Hollister, and West Los Angeles lines, a modification of the band spacings has been made for the lighter pressures in recognition of this strain, it being, however, ignored where spacings became less than 8 or 9 ins. On the St. Paul line, while it is not clear what general assumptions have governed the determination of the band spacings, the factors used indicate some recognition of these strains.

Any estimate of the strains due to the swelling of the staves under the maximum band spacing on the Provo line has been omitted. With a spacing of 18 ins., under a water pressure of 25 ft., a stave 1⅜ ins. in thickness, a pressure of 211 lbs. per lineal inch of band on the stave and a lack of hard cinching during construction, it is doubtful if such a strain exists to a considerable extent.

The Caldwell, or Phillis, pipe was a bold design as originally built, and is the only instance known to the author where pipe bands were actually broken by the swelling of the staves. The lumber used was Eastern Oregon mountain pine. A factor of only 1.64 seems to have

been used in the spacing. On the assumption that stave swelling produced a strain of 100 lbs. per square inch of contact between staves, doubtless a very conservative assumption with this class of lumber and size of pipe, and, with 6-in. band spacing, this band factor would be reduced to 1.30. Uncertainty as to the exact tensile strength of the material used, and the use of a shoe of the type shown in Fig. 4, with the probable resulting bending of the band under the nuts, is doubtless sufficient to account for the rupture of the bands. The pipe being of large diameter, the pressure of the bands upon the staves is seen to have been too low to admit of any further yielding and consequent partial release of the band strain.

The difficulty with this pipe is said to have been remedied by largely increasing the number of bands.

Compressive Resistance of Staves.—The highest degree of compressive strain between band and staves necessary for the maintenance of a tight pipe seems to have been used at Provo, and the results are instructive. The ready stripping of threads, due to poorly fitting nuts, prevented very hard cinching and satisfactory compression of the stave under the band. The timber was seasoned redwood. The pipe, a gravity line, is closed at the end, feeding direct into the distributing system. The use of hydrants for street sprinkling and other purposes caused a water pressure at the lower end of the line of 30 lbs. above the normal, making the pressure on the staves 290 lbs. per lineal inch of band under these conditions, or 1 550 lbs. per square inch of band contact. Leakage resulted, which was remedied by the addition of more bands, the re-cinching of those already on, and the partial prevention of the water hammer. The upper section of the pipe, not being subject to any considerable increase of pressure over that for which it was banded, has never given any trouble, although, as seen in Table No. 5, the normal pressure on the staves cannot be much less than 211 lbs. per lineal inch of band, or 1 126 lbs. per square inch of band contact.

On the Butte line the staves are depended upon to afford a permanent resistance against band pressure of not less than 900 lbs. per square inch of band contact. At Astoria the fir staves must resist a pressure of not less than 832 lbs. per square inch of band contact. Both of these lines have been unqualifiedly successful, and the latter line, especially, has from the first been remarkably free from leakage.

The former, by reason of the yet uncompleted condition of the storage dam which impounds the water from which the pipe is supplied, has not as yet been subjected to quite all the pressure for which it is designed.

In the smaller pipes subsequently built at Hollister, it will be noticed that much additional steel has been used in order to secure greater area of contact between bands and staves and thus keep the necessary compressive resistance of the staves down to from 600 lbs. to 686 lbs. per square inch. At the present time the same results might be achieved at considerably less cost by the use of bands of oval instead of round section. Such a course, as tending toward conservatism in a matter still somewhat imperfectly understood, and to partially nullify the effects of any lack of skill in erection, the author believes to be advisable in the design of the smaller sized pipes.

Construction Details.—There are but few structural details of this class of pipe in which there is sufficient diversity of practice to warrant any discussion. The butt-joint connection, the use or omission of a bead along one edge of the stave and the coupling shoes are the only features which need be considered.

Butt Joints.—It is the prevailing practice to make the connection between the butts of connecting staves by the snug insertion of a No. 12 or No. 14 steel plate about $1\frac{1}{2}$ ins. in width, and slightly longer than the width of the stave where inserted, the plate extending slightly less than half its width into each stave. For this, in some light pressure pipes, a plate of hard wood has been substituted. The objection to the latter lies chiefly in the difficulty of forcing a thick wooden plate into the adjoining stave sufficiently to secure a tight joint, and in securing the perfect fit evidently necessary. The existing general preference for the steel plate seems justified.

Plain vs. Beaded Edges.—The use of a bead on the stave edge is in no way vital to the success of a pipe, but seems to have strong points in its favor. If there is any irregularity in the edge of the plain stave, with band spacing suitable for light or moderate pressures, it is not possible and certainly not economical to induce sufficient fiber compression between the staves to take up those irregularities. By the use of a slight bead the compression due to cinching is at first concentrated on the bead, forcing it into the adjoining stave, and thus a uniformly tight seam is secured without the necessity for hard

compression throughout the entire contact area that exists in the other case. Furthermore the bead adds nothing to the cost of the stave.

Coupling Shoes.—Not a little ingenuity has been displayed in devising various types of coupling shoes, the desideratum being that with the least cost the shoe should have an ultimate strength fully equal to that of the band, a sufficient and properly distributed bearing surface to withstand high strains without destructive crushing of the

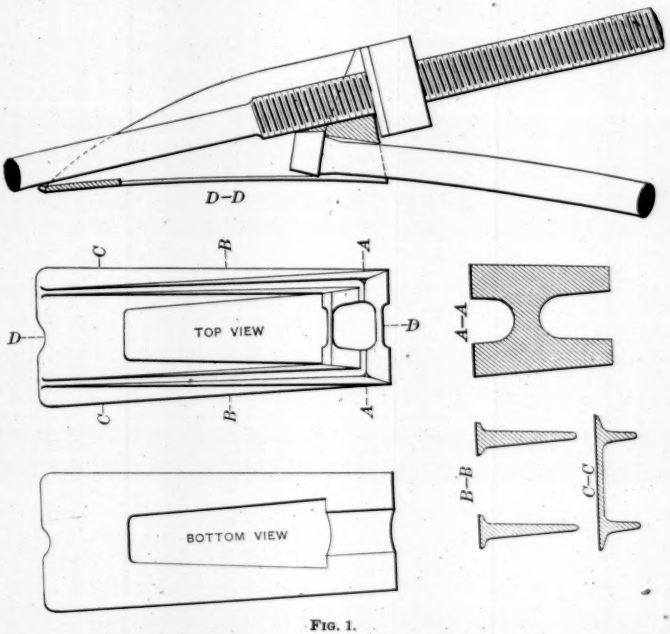


FIG. 1.

stave, and admit of speedy placing and the ready adjustment of the band during erection.

In the following discussion only those types are considered which have separate characteristics, are the better of those that have been used, and which have already met, or promise to receive, the greatest measure of favor.

The various devices in use may be divided, with reference to the manner of engaging the two ends of the band, into two general classes:

(1) Those which engage the two ends in the same transverse plane, as Figs. 1, 2 and 3.

(2) Those in which the ends are not engaged in the same transverse plane, as Fig. 4.

With reference to the materials of which they are made, they may be divided into:

(1) Forged, as Figs. 2 and 3.

(2) Cast, as Fig. 4.

(3) Malleable cast, as Fig. 1.

Fig. 1 is the general form of shoe that has been largely used by

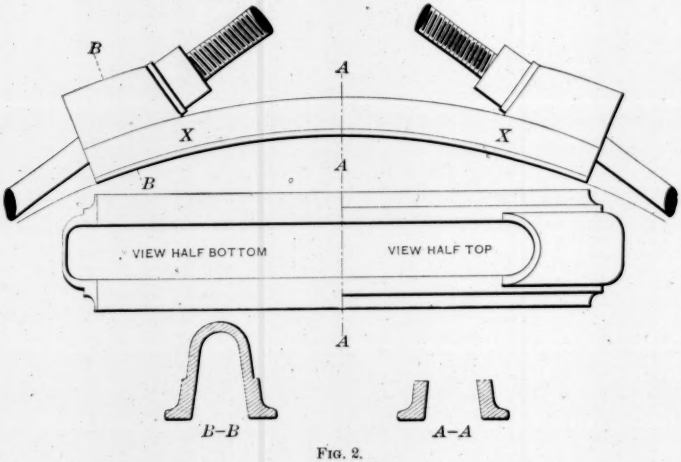


FIG. 2.

the Excelsior Wooden Pipe Company, it having been evolved from, and being representative of, the type originally used in the construction of the Denver lines. When properly proportioned this shoe admirably meets all the requirements for a successful connection, and, in the economical distribution of the material for resisting the strains imposed, leaves little to be desired, and possesses much to be admired. It will be noted that by the use of a T-head, the nut is brought as low as possible and the resulting moment tending to produce rotation about the center of the head bearing is thus reduced to a minimum. The length and ample bearing of the shoe enables the stave to resist this tendency without destructive pressure. The flanges on the lower

edges of the ribs afford bearing surface and give sufficient sectional area to resist the tensile strains here existing, while, in height, the ribs are well proportioned for transmitting the band strain to the staves. It will also be observed that the shoe permits the use of the wrench without interference. The feature most open to criticism in the design of this shoe as generally used is the tendency to make insufficient allowance in its weight for the deterioration of such light sections.

Fig. 2 shows a shoe of a type that has been used in a number of instances on the Pacific Coast. It is a forging from a rolled section. The length of the shoe in the rear of the nuts is wholly insufficient to

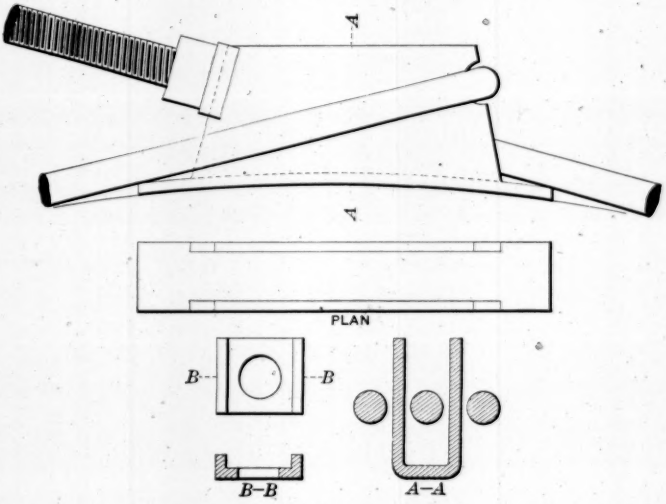


FIG. 3.

afford the proper degree of bearing surface on the stave to safely resist the pressure; and the bending strain at *X*, due to the depressing of the end of the shoe into the stave, can only be resisted by the use of a much heavier section than the tensile strains alone require, or prevented by giving more supporting area under the ends. It is evident that the fault lies in the use, unavoidable here, of low connecting ribs.

Fig. 3 is a forged shoe of the type used on the recently completed work of the Pioneer Power Company, at Ogden, Utah.* In most

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 270.

respects the form is evidently well calculated to economically resist the imposed strains; but, as it necessitates the use of a welded loop on one end of the band, the shoe can scarcely compare in economy with some of the other types, unless indeed the relative cost is very largely influenced in its favor by royalty charges on the other shoe. The details of the loop bearing on the shoe are perhaps not beyond

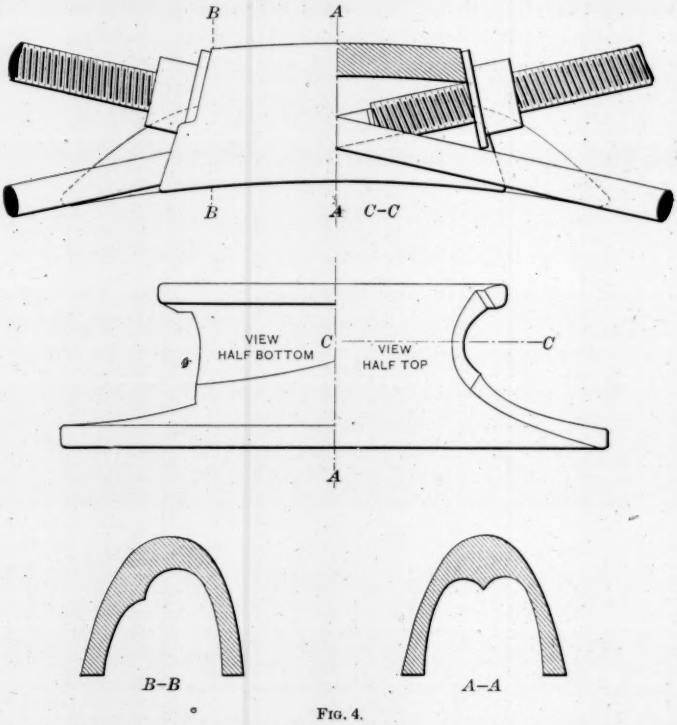


FIG. 4.

the suggestion of improvement, as there appears to be possible lack of stiffness, which, however, could be readily remedied by the insertion between loop and shoe of a properly designed compression lug of cast iron. It would also appear that the cost of the entire shoe might be somewhat reduced without sacrifice in other respects by modifying the design and making it of cast or malleable cast iron,

for which materials the general shape of the shoe, as affecting the strains, seems especially well suited.

Fig. 4 is of that class which does not engage the ends of the band in the same vertical plane. There being thus introduced a cross-strain or tendency to rotate about an axis vertical to the center of the shoe, it is difficult to devise a really efficient and economical design. It is usually sought to resist this tendency to rotate by means of the friction afforded by long tapering extensions at each end of the casting, as shown in the figure. This device has generally been found insufficient, hard cinching usually resulting in the breaking of these tail pieces or the rotating of the shoes and the bending of the bolts. These difficulties can, of course, be corrected by the use of very large, heavy shoes, but with a resulting cost much beyond that of other and better types.

This general style of casting seems to have been the first used in stave pipe construction with round bands, and has had quite an extended application in recent years, though frequently with unsatisfactory results. The particular design shown is that adopted in the construction of the Los Angeles outfall sewer, in the spring of 1894, and in detail is the best of its class that the author has seen. Nevertheless, it was found to possess the defect inherent in the type, many having broken under the reputed light and insufficient cinching used on that work.

In conclusion, it appears to the author that the essential requirements of a safe and economical design are much more successfully worked out in Fig. 1 than in any of the others.

The Economy of its Use.

In seeking to set forth clearly the real place of stave pipe in the economic design of pressure conduits, it is essential that the conspicuous types of pressure pipes be briefly considered in their relation to each other. The three varieties to be contrasted herein are stave, steel riveted and cast iron. Welded pipe is omitted because it occupies an intermediate position and does not affect the merits of the discussion. The comparison will be confined to the pressure range of practical stave pipe construction, say, not to exceed 100 lbs. per square inch, for which pressure each of the classes mentioned may be assumed to be wholly applicable.

Previous to argument, the relative standing of each, in the order of their merit with reference to the three most important qualifications, the author tabulates as follows:

	Cost.	Life.	Capacity.
(1)	Stave.	Cast iron.	Stave.
(2)	Steel riveted.	Stave.	Cast iron.
(3)	Cast iron.	Steel riveted.	Steel riveted.

If the quantity of water delivered, instead of the diameter, were made the common basis of comparison for the different pipes, the capacity would become a function of the cost, an aspect briefly considered later. For the present they will be considered separately.

Life.—Passing temporarily over the question of cost and taking up that of comparative duration, it may be assumed at once that cast iron should be given the preference, not, indeed, as conclusively proven by experience, but as in every way probable, since the bands of the stave pipe, excepting, perhaps, in the larger sizes, can hardly be supposed to have the same degree of endurance as the thick shell of the cast pipe.

In placing steel pipe last, the author is influenced by the following considerations:

(1) That the sheet metal from which the pipe is made is of less thickness than the diameter of the stave pipe bands, a condition always existing in steel pipes subject to the comparatively light pressures here considered.

(2) That the same degree of care in securing protection of the metal can be exercised in both cases.

(3) A properly proportioned steel pipe does not eventually fail by bursting, but by the formation of rust holes and attendant leakage. The considerable multiplication of such holes soon causes a new pipe to be cheaper than the necessary repairs, even though $\frac{995}{1000}$ of the metal in the pipe may be as good as when it was laid. The life of a steel pipe, therefore, without material error, may be said to be measured by the life of its weakest spots.

(4) The life of a stave pipe is probably determined by the life of the bands; and the life of the bands is determined, not by the time necessary for the formation of local rust pits, even though they are very deep, but by the reduction of the entire band section until its ultimate strength is exceeded by the strain, say to less than one-fourth

its original section. It seems clear that the time requisite to accomplish this degree of deterioration in the sections generally used must be many times that necessary to accomplish the destruction of the average riveted pipe.

(5) Many cases on the Pacific Coast might be cited where steel pipes of the lighter gauges, though carefully coated with asphalt, and apparently laid with reasonable care, have rusted through in from eighteen months to three years, and have been shortly afterward abandoned.

(6) Should a perfect and reasonably cheap protective coating for steel ever be discovered—an improbable assumption—no single class of pipe would have much advantage over another in respect to durability.

(7) It is certainly in accord with universal engineering experience to assume that wood, under the conditions prevailing in proper stave pipe construction, is practically indestructible ; and much direct evidence in harmony with this assumption has accumulated from the many instances of the use of stave and other wood pipe constructions which have done service in America for from twenty to fifty years, and, so far as the author knows, without an instance of failure from the decay of the staves, where the proper requirements were observed.

The author does not wish to be understood as intimating that steel pipe is always short lived. On the contrary, many such pipes, where conditions are favorable, have been doing excellent service for many years; but the exceptions are instructive, and fully vindicate the position assumed by many engineers, who, for permanent work, have recognized the necessity for a considerable thickness of metal, independent of the demand for a sufficient factor of safety against bursting, and who accord little favor to pipes of very light gauge.

The foregoing seems to the author to be ample justification for placing the stave pipe far in advance of the steel pipe for durability under average conditions.

Capacity.—The past few years have added much to the knowledge of engineers concerning the carrying capacity of cast-iron and particularly of steel riveted pipes of various diameters. The relative values of each class, as economic carriers of water, are better understood than ever before. The general result of experiments on cast-iron pipe has been to prove the value of the Kutter formula in showing the value of

"n" to be practically a constant for any given diameter, degree of surface roughness and velocity of flow. In the case of steel riveted pipes, whether built with cylindrical or taper joints, with few exceptions, the results indicate that the Kutter formula does not apply, and that for the ordinary ranges of velocities, say from 3 to 6 ft. per second, for clean pipes the coefficient c in the formula $v = c\sqrt{r\,s}$ is nearly a constant for all diameters; and if the experiments on the Portland, Ore., pipes are excepted, because of inability to harmonize them with others, it is not clear that the results are much affected by variations in the thickness of the plates, within the range of ordinary usage.

These experiments, while still leaving much to be desired before attempting the formulation of any positive rules for the exact preliminary determination of the rate of flow in any given case, a result which the author believes will not be accomplished save within limits of very narrow application, because of the usual unavoidable lack of similarity in the influencing physical conditions, are sufficient to accomplish the complete upsetting of many previously conceived ideas regarding the laws of flow in this class of pipe. The failure of "c" to increase as the diameter increases places the value of this pipe, as an efficient water carrier, at least for the large diameters, in a relation to pipes having approximately smooth interiors, totally different from that which it has previously been accorded.

The dissipation of the once prevalent idea that wrought-iron and steel pipes are not subject to tuberculation, and the later conviction that in this respect they have little or no advantage over cast iron have also done much to make possible a better estimate of the real merits of this pipe.

The author regrets that as yet so few experimental determinations of the carrying capacity of stave pipe under varying conditions of diameter and grade have been made. In engineering literature he knows of only four in which the proper conditions have been sufficiently observed to render them worthy of mention. They are as follows:

(1) An experiment on a long line of 30-in. pipe at Denver, briefly mentioned by J. D. Schuyler, M. Am. Soc. C. E.* In this case, with a mean velocity of about 2.33 ft. per second, the derived value of n in the Kutter formula was found to be a little less than 0.010.

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 144.

(2) An experiment by the author on an 18-in. pipe,* in which the value of c in $v = c\sqrt{r\,s}$ was found to be 133, with a mean velocity of 3.60 ft. per second. This result corresponds to a value for n in the Kutter formula of a little less than 0.010.

(3) Experiments on a section of the 6-ft. pipe of the Pioneer Electric Power Company, at Ogden, Utah, described by Charles D. Marx, M. Am. Soc. C. E.; Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins.† These results gave average values for c as follows:

Velocity in feet per second.	Corresponding value of c.
1.0	100
2.0	115
3.0	122
4.0	125

The values of n corresponding to these results vary from 0.012 to 0.015.

(4) Experiments by the author on a line of 14-in. pipe,‡ in which the values of c averaged as follows:

Velocity in feet per second.	Corresponding value of c.
0.7	102
1.2	111
1.5	112

The values for n corresponding to these results vary from 0.0107 to 0.011. The velocities, it will be observed, are all very low.

The results of the Ogden experiments are so much at variance with the great preponderance of credible evidence bearing on this question, that, without wishing to seem to discredit them, the author awaits further verification of the results before accepting them as conclusive.

In Table No. 6, showing the values of c in the formula $v = c\sqrt{r\,s}$, for clean pipes of staves, cast-iron and lap-riveted steel, the values for the stave and cast-iron pipes are deduced by aid of the Kutter formula, assuming a value for n of 0.010 for the stave pipe, of 0.011 for the cast pipe when clean, and 0.014 when tuberculated, a velocity of flow of 3 ft. per second being assumed. For steel riveted pipes the uniform value of 111 is given for c for all diameters, and a

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 26.
† *Transactions*, Am. Soc. C. E., Vol. xl, p. 471.
‡ *Transactions*, Am. Soc. C. E. Vol. xl, p. 545.

deduction of 20% is made in determining the values for tuberculated pipes.

The calculated values are for the most part interpolated from tables, and may be slightly in error in the last figure.

TABLE No. 6.—Values of c in the Formula $v = c\sqrt{r\,s}$ for Clean Pipes of Wood, Cast Iron and Riveted Steel; and for Cast-Iron and Steel Riveted Pipes when Tuberculated.

Diameter.	Stave.	Cast Iron.		Steel Riveted.	
	$n = 0.010$.	Clean. $n = 0.011$.	Rough. $n = 0.014$.	Clean.	Rough.
12	119	105	77	111	89
18	130	117	86	111	89
24	137	123	92	111	89
30	143	129	97	111	89
36	148	133	101	111	89
42	152	136	105	111	89
48	154	138	108	111	89
54	157	140	110	111	89
60	159	142	112	111	89
66	161	144	114	111	89
72	163	146	116	111	89

If the discharging capacity of the stave pipe of any diameter is designated as 100, the relative discharging capacities of the others will be as follows:

TABLE No. 7.—Relative Discharging Capacities of Pipes, Stave Pipe Being Taken as 100.

Diameter.	Stave.	Cast Iron.		Steel Riveted.	
		Clean.	Rough.	Clean.	Rough.
12	100	88	65	93	74
18	100	90	66	86	69
24	100	90	67	81	65
30	100	90	68	78	62
36	100	90	69	75	60
42	100	90	69	73	58
48	100	90	70	72	58
54	100	89	70	71	57
60	100	89	70	70	56
66	100	90	71	69	55
72	100	90	71	68	54

If the premises are correct, and they are believed to be in harmony with present knowledge, these results are of a startling nature. Clean cast-iron pipes would seem to have about 90% of the carrying capacity of stave pipes, while if seriously tuberculated, a condition which usually prevails after a few years of use, they discharge only about two-thirds as much as stave pipe of the same size.

The steel pipe discharges when clean from 93% in case of a 12-in. pipe to 68% in case of a 6-ft. pipe of the amount which might be expected from stave pipes of the same sizes, while if the steel pipe is tuberculated to an extent that may readily occur in from ten to fifteen years of use, these discharges may fall to 74% and 54% respectively.

Omitting other considerations, is not the value of a pipe line investment proportional to its delivering capacity? Do not these percentages, then, represent approximately the relative values of these different classes of pipe as investments?

Cost.—Variability in market quotations on materials, freight rates, wages, the effect of geographical location on these, the conditions affecting haul and the manner of executing the work make it impossible to deduce an accurate comparison of cost which would admit of general application. For present purposes, however, it is not necessary to attempt more than to convey a general idea of the order of the respective costs of the three classes of pipe considered, at the terminal points designated. For this purpose Tables Nos. 8 and 9 have been prepared with considerable care.

It is assumed that the stave pipe is designed in accordance with the formulas previously deduced, an assumption which somewhat increases the cost, particularly for light pressures, above that which would result were the bands spaced in accordance with the formula more commonly used.

The steel pipe is supposed to be double riveted on the straight seams and single riveted on round seams, as ordinarily built, and coated with asphalt. The mill price for sheet steel is taken at from 1.60 for No. 14 plate to 1.25 for thicknesses greater than No. 8.

The cast pipe is supposed to be proportioned as to thickness by the formula of the Warren Foundry, and the prices assumed per ton are $19 at Chicago and $26 at San Francisco, the latter figure being the lowest yet quoted there.

TABLE No. 8.—Comparative Cost of Pipe at Chicago.
(Including Laying, but not Hauling.)

Nominal diameter.	Stave.				Steel Riveted.								Cast Iron.			
	25-ft. head.	50-ft. head.	100-ft. head.	200-ft. head.	No. 14, B. W. G.	No. 12, B. W. G.	No. 10, B. W. G.	No. 8, B. W. G.	No. 6, B. W. G.	¼ in.	⅜ in.	⅝ in.	25-ft. head.	50-ft. head.	100-ft. head.	200-ft. head.
12	.42	.49	.63	.85	.32	.38	.44	....	....	....	....	....	.73	.77	.84	1.00
18	.69	.80	1.02	1.46	....	.57	.65	.78	.98	....	....	....	1.29	1.35	1.46	1.70
24	.79	.91	1.14	1.61	....	....	.85	1.04	1.28	1.55	1.99	....	1.91	2.00	2.18	2.55
30	.96	1.12	1.44	2.06	....	....	....	1.27	1.59	1.93	2.46	3.04	2.67	2.80	3.07	3.61
36	1.19	1.40	1.82	2.65	....	....	....	1.55	1.93	2.30	2.92	3.58	3.47	3.67	4.06	4.85
42	1.40	1.68	2.23	3.33	....	....	....	1.61	2.18	2.66	3.37	4.12	4.42	4.69	5.22	6.28
48	1.55	1.85	2.46	3.67	....	....	....	....	2.48	3.03	3.83	4.66	5.50	5.84	6.53	7.92
54	2.23	2.62	3.43	5.02	....	....	....	....	2.80	3.41	4.29	5.21	6.65	7.10	8.00	9.78
60	2.85	3.35	4.37	6.40	....	....	....	....	....	3.79	4.75	5.74	8.04	8.63	9.80	12.13
66	3.21	3.81	5.00	7.38	....	....	....	....	....	4.35	5.21	6.29	9.51	10.16	11.55	14.05
72	3.65	4.38	5.88	8.73	....	....	....	....	....	4.52	5.66	6.83	11.32	12.00	13.26	16.00

TABLE No. 9.—Comparative Cost of Pipe at San Francisco.
(Including Laying, but not Hauling.)

Nominal diameter.	Stave.				Steel Riveted.								Cast Iron.			
	25-ft. head.	50-ft. head.	100-ft. head.	200-ft. head.	No. 14, B. W. G.	No. 12, B. W. G.	No. 10, B. W. G.	No. 8, B. W. G.	No. 6, B. W. G.	¼ in.	⅜ in.	⅝ in.	25-ft. head.	50-ft. head.	100-ft. head.	200-ft. head.
12	.37	.45	.61	.94	.45	.52	.60	....	....	....	....	....	.94	.99	1.09	1.31
18	.58	.71	.96	1.47	....	.77	.88	1.04	1.30	....	....	....	1.65	1.73	1.89	2.21
24	.76	.96	1.35	2.12	....	....	1.01	1.35	1.67	1.97	2.51	....	2.50	2.62	2.86	3.37
30	.83	1.03	1.43	2.23	....	....	....	1.64	2.04	2.45	3.10	3.79	3.52	3.70	4.07	4.81
36	1.02	1.23	1.83	2.90	....	....	....	1.80	2.44	2.92	3.67	4.49	4.60	4.87	5.40	6.50
42	1.23	1.58	2.28	3.69	....	....	....	2.27	2.80	3.37	4.24	5.12	5.87	6.25	6.97	8.43
48	1.34	1.73	2.51	4.07	....	....	....	....	3.14	3.80	4.78	5.78	7.33	7.80	8.73	10.64
54	1.94	2.45	3.49	5.52	....	....	....	....	3.55	4.32	5.37	6.47	8.86	9.47	10.69	13.15
60	2.44	3.09	4.39	7.00	....	....	....	....	....	4.87	5.95	7.24	10.73	11.54	13.14	16.33
66	2.79	3.57	5.14	8.26	....	....	....	....	....	5.43	6.54	7.81	12.70	13.69	15.45	18.92
72	3.22	4.15	6.04	9.78	....	....	....	....	....	5.90	7.09	8.50	15.12	16.05	17.78	21.53

The figures are supposed to include only the principal common items of expense, with no profit to the contractor. They are therefore, perhaps in every case, somewhat below probable cost, and are intended for use by way of comparison only.

A contrast of actual quotations for particular cases would in almost every instance show a greater advantage to the stave pipe than the tables indicate. Especially would this be true where a considerable haul would be necessary, or where the work is far removed from pipe shops and of insufficient magnitude to justify manufacturing the riveted pipe on the ground, or where the site is difficult of access. The tables indeed assume the most favorable conditions for the use of the steel riveted or the cast pipes.

Conclusion.—If, then, stave pipe stands unquestionably first in point of both first cost and carrying capacity when contrasted with the two other classes of pipe usually considered, and second only to cast iron in length of life, does it not logically follow that it is destined to a greatly extended use? and is it not clear, that though the great saving in first cost incident to local causes has fostered its rapid introduction in the West, yet the field of its economic usefulness is by no means compassed by such narrow limits? If these queries are answered in the affirmative, is it not deserving the careful consideration of engineers who conscientiously and studiously seek a wise economy in the design and execution of hydraulic works?

DISCUSSION.

Mr. Tillson. GEORGE W. TILLSON, M. Am. Soc. C. E.—Mr. Patten's* statement shows that wood which is alternately wet and dry, rots very rapidly in a climate like that of Wyoming. In the case he describes, where the pipe was running at times not much more than half full, such results would naturally be expected. The author evidently appreciates fully the danger in that respect, because he specifies that the staves shall be so thin that the water will thoroughly permeate them.

The only explanation the speaker can give for this rapid decay is that the wood is dry so long that it becomes thoroughly seasoned, or thoroughly free from moisture, and then, when rain occurs, such wood is in the right condition to absorb a large amount of moisture. In a short time it becomes dry again, and so goes on in that way alternately thoroughly wet and thoroughly dry.

Several instances of such decay have been noticed by the speaker. In Omaha, Neb., a lot was being graded, and, in order to work grading machines to advantage, it was necessary to grade the earth over an adjacent wooden sidewalk. When the work was completed the sidewalk was cleaned off, and, although the earth had only been on the walk two or three months, the planks had decayed appreciably.

In wooden block pavements laid in Omaha, although the spaces between the blocks were filled with sand, and the interstices with tar, in a year and a half or two years, cedar blocks began to decay around the edges, and in three years cypress blocks were almost entirely gone.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—Questions as to the durability and capacity of stave pipe will have to be settled by a little longer experience than has as yet been obtained. Cast iron and steel are more expensive on the Pacific slope than in the East, while wood is cheaper. For that reason they have manufactured a good deal of stave pipe, and have had more experience with such pipe in that country than in the East; and while it is somewhat courageous in the author to take the position that wood will last longer than iron, yet there is much truth and force in his reasons. In California the speaker has seen wood pipe that had been in the ground thirty years, and which was as good as the day it was put in. He has also seen wrought-iron pipe hardly $\frac{1}{8}$ in. thick that had been used in the ground, and, afterward for two years had been lying in the field, and on which scarcely any rust could be seen, merely a discoloration.

The speaker believes that there is a considerable future for stave pipe, provided that great care is taken in the selection of the materials, in their manipulation and in the construction. The question of rot

* See p. 65.

is answered in this way: If the pipe is painted inside, the wood will almost surely rot out; it should not be painted or covered with any oily material, because the water which is in the pipe under pressure should go through the wood, and keep it thoroughly wet all the time, thus preventing rot. The pipe before mentioned by the speaker had not been painted inside; he was shown pipes that had been so painted, and they had rotted in a few years. It is necessary, of course, to have the water under pressure constantly, so that it will ooze through the pores of the wood, and keep it in a condition similar to that which protects wood under water. The loss of water by the oozing is insignificant. Mr. Hering.

It has been found that bands having a circular or elliptical section are very much better than the flat bands formerly used, because for the same quantity there is a smaller surface of metal exposed to corrosion. Of course, a great deal depends upon the care with which the material for the bands is made, and then upon the care with which the bands are coated and fastened around the pipe. The speaker saw in Tacoma some of these bands broken while they were being put around the pipe, thus showing that they had been improperly made. Great care must be taken in all the details of manufacture in order to make a successful stave pipe. The speaker believes, however, that if the work is well done, the Pacific Coast, at least, has a means of laying and maintaining water pipes at a great deal less expense than if either steel or cast iron were used.

The wrought-iron pipe before mentioned was used by the Spring Valley Water Company in San Francisco. The chief engineer there maintained that wrought iron was better than steel, because, if the material was sufficiently manipulated, so that practically no uncombined carbon was left therein, there would be less occasion for rust. The uncombined carbon in the steel and in the cast iron, in connection with the carbonic acid in the air and in the water, causes electrical action, and therefore corrosion. The specifications required a very severe treatment in the manufacture, rolling over and over many times, so that a high degree of homogeneity was obtained, and electrical action in the material was practically prevented. Cast iron apparently does not corrode so much as wrought iron, but this is due to the fact that it is protected on the outside by magnetic oxide. If that is removed, cast iron will corrode more quickly than homogeneous steel or wrought iron.

In the speaker's opinion there is no question but that the carrying capacity of a wooden pipe, made of staves which have been planed, will be greater than that of a riveted steel or cast-iron pipe. There have been so many observations to confirm this that it cannot be questioned so long as the pipe is kept in good condition. However, some change in that condition may take place. For instance, a slimy coating may

Mr. Hering. become attached to the wood, or plants grow thereon, causing a great resistance and consequently a diminished flow.

In reply to a question the speaker stated that he had never heard of any trouble caused by any chemical action of the iron in eating the wood away.

Mr. O'Rourke. J. F. O'ROURKE, M. Am. Soc. C. E.—The author believes that it is profitable to limit the use of stave pipe to pressures lower than 100 lbs. per square inch. In a 6-ft. pipe, 100 lbs. pressure would be equivalent to the maximum strain allowable on a steel shell ¼ in. thick, or about 15 000 lbs. per square inch. In a 6-ft. pipe, omitting the strains due to cinching and to the swelling of the staves, and considering only the strain caused by a pressure of 100 lbs. per square inch, the bands would be so numerous that their cost would probably be as great as that of a cast-iron pipe. The author, therefore, should fix more closely the economical limit as to the diameter of the pipe and the pressure at which it can be used.

The limit of pressure fixed by the author seems to be too great, and, so far as durability is concerned, the speaker takes issue with him, because of the difficulty in protecting the bands and because such pipes are not yet cheaper or more economical than cast-iron pipe under certain conditions. They may be cheaper than cast-iron pipe in first cost, but they are not as long lived.

The author should be a little cautious in stating that steel bands, hammered around the wood and buried in the ground, will last longer than steel pipe treated in the shops.

A few years ago the speaker had occasion to remove from Broadway, New York City, some wooden pipe, which had been put down by the old Manhattan Water Company. It had been in the ground in the neighborhood of a hundred years, and was a little soft, but not at all decayed. Its condition was just what would naturally be expected in wood that had been so long in a damp, but not wet, condition. That is, the condition of wood which has been in comparatively moist earth, and which has not been exposed to the air.

A similar case of old wood, under like conditions, was found in the summer of 1898, in an old building at the corner of Gold Street and Maiden Lane, New York City. The bottom of the wall consisted of oak mud-sills, about 5 or 6 ft. long and 6 ins. thick. These planks were, perhaps, less than a foot above the water and were quite sound. On top of the mud-sills, hewn timber, about 24 ins. wide and 10 or 12 ins. thick, was laid the full length of the wall. This timber was fully 18 ins. above the ground-water. It was simply in moist clay and was perfectly sound, although the building was more than a hundred years old.

So far as stave pipe is concerned, when it is well buried in moist earth the speaker does not think there is any serious danger of the

wood rotting. The danger is confined entirely to the corrosion of the bands, as the author states. Mr. O'Rourke

It is stated in the paper that the thickness of the staves should be limited so that they will become thoroughly water-soaked. The speaker believes this dimension to be very large.

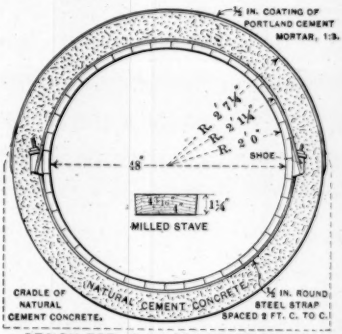
In the summer of 1898, the speaker put down some wooden caissons which were similar to stave-pipe in construction, except that the rings were on the inside. The staves were of yellow pine, very rich in pitch, and were $3\frac{1}{2}$ ins. thick. The water in the ground around the caissons was kept below its original level by pumping, to about 6 ft. from the surface of the cellar. The caissons were sunk about 30 ft. deeper than this, and some of them had been in the ground for nearly a week before the pig iron and air shafts were taken out and the interiors made visible. It was found that the water had penetrated the $3\frac{1}{2}$ ins. of rich pine, and that it indicated distinctly the outside water-level.

The speaker thinks, therefore, that the question of making the staves thin enough to allow the water to soak to the outside need not give much trouble. The author recommends staves $1\frac{3}{8}$ ins. or 2 ins. thick to permit of complete saturation; in the opinion of the speaker the staves might be a foot thick, if such a thickness were ever necessary, and that still the water would soak through to the outside and always keep the timber wet while the pipe was full.

The speaker considers that such an excessive thickness would never be called for, and only mentions it to show that the saturation of the staves is certain in any event, so long as one side is in contact with water.

JAMES C. MEEM, Assoc. M. Am. Soc. C. E.—There are two points of weakness in the construction of stave pipe, which appear to have been met, and which are noted in the discussion, namely, the tendency to rot on the outside, and, in pipes of large diameters and made of thin staves, the tendency to distortion. Mr. Meem.

Some time ago the speaker, under C. H. Myers, M. Am. Soc. C. E., assisted in designing a 4-ft. sewer pipe intended to run under pressure. Comparative estimates as to the cost of cast-iron pipe and wood-stave, concrete-cased pipe were made.



DESIGN FOR 48 IN. CONCRETE-CASED WOOD-STAVE SEWER.

FIG. 5.

The cost of the former was found to be about $10.50 per foot, while the latter, to be built substantially as shown in Fig. 5, was estimated to cost about $5.75 per foot. Fig. 5 is a section of the

Mr. Meem. design, and would seem to overcome the two points of weakness noted. The shoe was substantially the same as that shown in Fig. 3, except that it was to have been made of cast iron. The rest of the design is self-explanatory.

Where the soil was soft or marshy, this sewer was to have been built on grillage, and, in that case, the design included a cradle of natural cement concrete conforming to the dotted lines shown in the figure, the cradle and concrete invert being, of course, merged into one. The cost of a sewer of this design on grillage of solid double rows of 2-in. planking at right angles was estimated to be about $8.00 per foot in this specific instance.

Unfortunately for experience this pipe has never been constructed, but it is believed that for certain purposes it would prove to be the most economic type of construction. The old City of New York has built a great deal of wood-stave sewer pipe; Brooklyn has also built several sewers of this material in the last two or three years, as it is especially well adapted to outfalls through crib-work and piers.

The speaker believes that these sewers have always been satisfactory, but he can only speak certainly of those in Brooklyn, and they have not yet been down long enough to give any indication as to their ultimate endurance. The tendency to rot is largely overcome by creosoting, and the tendency to distortion is prevented by making the staves quite thick.

Mr. Skinner. F. W. SKINNER, M. Am. Soc. C. E.—Although the limits of economy of stave pipe and steel pipe are being contrasted, the speaker thinks there is some comparison between the economy of stave pipe and bored wooden pipe. He had occasion not very long ago to visit an establishment where bored wooden pipe is manufactured, and was quite impressed with the great economy of production. Large logs were turned to a suitable exterior diameter, and then the core was taken out so as to preserve the heart of the timber. The core was about 8 or 9 ins. in diameter, and sufficed for a smaller pipe, and was in turn counter-bored. The pipes were reinforced by the well-known method of winding, under tension, a spiral of thin flat metal around them. The ends were counter-bored slightly and connections made by inserting wooden thimbles. The process is extremely cheap as regards first cost, and for small lines under moderate pressures should give economical results.

CORRESPONDENCE.

HENRY B. PATTEN, M. Am. Soc. C. E.—The City of Cheyenne, Wyo., has built, altogether, under the Allen Patent, 10 259 ft. of stave pipe, as follows: Mr. Patten.

In 1888, 1 892 ft. of 30-in. pipe.
In 1889, 6 333 ft. of 30-in. pipe.
In 1889, 2 034 ft. of 20-in. pipe.

These pipes were planned as a conduit for water running under little or no head. A portion of the pipe was laid under circumstances such that it was only partly full of water during the greater portion of each year. As might have been expected, the portion so constructed decayed rapidly, and failed entirely in a few years.

Of the 30-in. pipe built in 1889, 2 000 ft. were replaced by vitrified earthenware pipe in 1896, after seven years' use. In 1897, 2 200 ft. more were replaced, having given only eight years' service. It is expected that it will soon be necessary to replace 2 133 ft. more of the 30-in. pipe, which is the remainder of the 30-in. pipe laid in 1889.

When the old pipe was removed, a considerable growth of grass roots was found inside. This is an impediment to the flow of water that should be considered as a partial offset to the tuberculation found in cast-iron pipe. It was also found that the pipe was distorted from the circular section. In some instances the cross-section was found to be quite elliptical, the original 30-in. circular pipe being found to be only 26 ins. in vertical diameter, while the horizontal diameter was nearly 34 ins.

In the writer's observation of the old pipe as it was removed, the following facts were noted:

First.—The decay of the pipe was greatest on the upper portion of the line where it carried the least water.

Second.—That pipe which had the least covering, and thus was nearest to the surface of the ground, was decayed the most.

Third.—The outside of the pipe, on the upper portion of its circumference, was decayed more than any other part of the pipe.

Fourth.—With old stave pipe, there may be, occasionally, a material resistance to the flow, due to the presence of grass roots which may have forced themselves through the moist staves or through the joints.

Fifth.—For the theoretically circular stave pipe, there is a possibility of distortion into an oval or elliptical cross-section, which will decrease the sectional area sufficiently to cause a diminution of the discharge.

F. W. D. HOLBROOK, M. Am. Soc. C. E.—At the eastern end of the Hoosac Tunnel, in 1865–1867, the air compressors, which furnished power for the drills, were operated by turbine wheels. The penstocks, Mr. Holbrook.

Mr. Holbrook. through which the water was conveyed from the canal to the wheels, were continuous wooden tubes, made of staves of variable length, with radial edges and concentric faces. The staves were held together by circular metal bands, spaced in accordance with the strains imposed. The penstocks were about 40 ft. long and about 5 ft. in diameter. Such construction was well known and in general use in western Massachusetts at that time, and therefore dates back to a much earlier period than is generally supposed.

Mr. LeConte. L. J. LE CONTE, M. Am. Soc. C. E.—The author is evidently strongly impressed with the special advantages of stave pipe, as compared with cast-iron and riveted pipe, within certain fixed limits.

The writer is quite willing to concede many of the important advantages claimed, provided the limit of pressure does not exceed 100 lbs. per square inch, as assumed in the paper. He takes exception, however, to the claim that the velocity coefficient c for stave pipe gradually increases with increase in diameter, and finally attains the extraordinary value of 163 for the 72-in. pipe. There is no proper authority for such a high value of c. With stave pipe, the highest value for c yet obtained by competent authority, and on a scale sufficiently large to inspire confidence, is about 143.

Of course, it is quite possible that future experience with larger pipes may develop higher values, but the Ogden experiments with the 72-in. pipe certainly do not furnish evidence upon which to base any hopes in this direction.

The writer strenuously objects to the palpably unfair consider ation given to riveted pipes in the paper. As pipe of this kind has been in constant use on the Pacific coast for the past thirty-five years, and on a very large scale, the writer is able to speak with confidence.

The life of any one of the three classes of pipe under consideration will depend chiefly upon the character of the soil in which it is buried. In ordinary soil, free from alkaline ground waters, riveted pipe, properly coated, will certainly last twenty-five years, as instanced to the writer's personal knowledge, by numerous examples. The following table from his memorandum book shows the life of some of the thinner grades of pipe:

Diameter, in inches.	B. W. G.	Year laid.	Year taken up.	Age, in years.	Head of water, in feet.
8	18	1874	1895	21	135
10	16	1865	1895	30	175
6	16	1873	1894	21	200
8	14	1865	1895	30	175
12	14	1868	1898	30	200
24	10	1872	1897	25	200

Mr. LeConte.

All these pipes were taken up, samples were taken and observations on the same duly recorded. The old pipes were then relaid under lighter pressures, and are now doing fairly good service.

Tuberculation is so rare on the Pacific coast that engineers, as a rule, give it little or no consideration in fixing the sizes of pipes. The waters being nearly always hard, the salts in time form a thin incrustation on the inner surface of the asphalt coating, and so far there has been no authentic case reported where c has materially diminished with the age of the pipe. Tuberculation is only troublesome where the waters are soft.

The writer particularly objects to the statements of comparative values of c in Tables Nos. 6 and 7. In Table No. 6 the value of the coefficient for clean steel-riveted pipe of all diameters and thicknesses is given as 111, which is clearly an error, and the coefficient for tuberculated 72-in. riveted pipe is given as 89, or the same as that for a tuberculated 20-in. cast-iron pipe. Comment is unnecessary.

It therefore follows that Table No. 7, giving the relative discharging capacities, is equally objectionable. The tables of comparative cost (Nos. 8 and 9) are replete with comparisons unfair to riveted pipe. For example, take the 30-in. pipe at San Francisco, Table No. 9. The cost of stave pipe, 30 ins. in diameter, capable of sustaining a 200-ft. head of water (a limit of comparison of the author's own selection), is given as \$2.23 per lineal foot; while the cost of $\frac{3}{8}$-in. riveted pipe 30 ins. in diameter, capable of sustaining a 200-ft. head of water, is given as \$3.79 per lineal foot. The author omits to mention the important fact that this $\frac{3}{8}$-in. double-riveted pipe will safely stand a 900-ft. head of water.

$$P = \frac{16\,000 \times t}{r}, \text{ or } P = \frac{16\,000 \times \frac{3}{8}}{15} = 400 \text{ lbs.} = 900 \text{ ft.},$$

in which P = the pressure in pounds per square inch,

t = the thickness of the shell in inches,

and r = the radius of the pipe in inches. The author, therefore, has, in this 30-in. pipe, four times the amount of steel that the case calls for—and so on through the whole table.

The strong points in favor of riveted pipe are economy in first cost, and durability—it lasts at least twenty-five years. In California, engineers have never claimed a high value for c for riveted pipe, as compared with cast-iron pipe of the same diameter. The value of c for large sizes is found to range from 115 to 120, and for small sizes from 100 to 110. These values are further modified by the percentage of heavy plate iron there may be in the pipe line. This factor makes the determination of the correct value of c, for any proposed pipe line, a very complex problem, so much so that it has become a maxim, so to speak, that every riveted pipe line has its own peculiar coefficient.

Theoretically, the stave pipe should certainly develop a high value

Mr. LeConte. for c, but this class of pipe has been in use only eight years, and there is much yet to be learned about it.

The sphere of its usefulness is so limited in extent that its economic application will be confined almost entirely to special cases where the pressures are low. For all higher pressures riveted and lap-welded pipes are the only kinds which are fit for the services they are called upon to perform.

Mr. Henny. D. C. HENNY, M. Am. Soc. C. E.—The author has distinguished three distinct sources of strain in the band of a stave pipe, and the formulas for the determination of this strain express the effect of each source separately, $P f R$ being the strain from water pressure, $X = \frac{3}{2} P f t$ the strain from cinching, and $E'' t f$ the strain caused by the swelling of the wood, all of which are added together to establish a value for S, the least permanent band strain in a tight pipe, as in the author's formula (1).

The writer cannot admit the correctness of this reasoning, because the interdependence of these strains, to which attention has been directed by the author himself on page 29, precludes the determination of the final strain by simple addition. Nor has the author made fully clear what was meant by X, whether the strain in the band immediately after back-cinching, or immediately before the pipe is filled with water, or whether it represents some remaining influence in the final strain of the cinching of the band. In any case there is no evidence why it should be in any way dependent on the water pressure P, and the value assigned to X by the author seems to be purely arbitrary.

The writer believes it to be self-evident that when there is no pressure on the pipe the strain in the band is balanced by the compressive lateral strain in the staves. Clearly, assuming the staves to have no compressive strength whatever, no amount of cinching could produce any strain in the band. When there is water pressure on the pipe, the lateral strain in the wood must balance all the excess in the band strain over that due to water pressure, otherwise motion would ensue until a balance is established.

Calling y the average compressive lateral strain in the staves per square inch of compressed section when the pipe is under a pressure P, then

$$S = P f R + f t y \ldots\ldots\ldots\ldots\ldots\ldots \quad (a)$$

The author is correct in reasoning that the ultimate swelling force of wood is the measure of its ultimate resisting force when saturated, hence the maximum permanent value of y is E'' and

$$S_{max} = P f R + f t E'' \leqq (R + t) E \ldots\ldots\ldots\ldots \quad (b)$$

TABLE No. 10.—Strains in Bands on 10-In. Redwood Stave Pipe, Mr. Henny. in Pounds.

Date.	Gauge Readings in Pounds.								
	0	5	10	15	20	30	40	50	60
Bands: 10 ins. center to center.									
Jan. 31st, 1896.	4 300	.....	.....	.....	.....	.....	.....	.....	.....
" 31st, 1896.	3 700	3 800	3 850	3 925	4 100	.....	.....	.....	.....
Feb. 1st, 1896.	2 600	.....	.....	.....	.....	.....	.....	.....	.....
" 3d, 1896.	2 300	.....	2 500	.....	.....	.....	.....	.....	.....
" 4th, 1896.	.....	2 150	.....	.....	.....	.....	.....	.....	.....
" 5th, 1896.	.....	2 100	.....	.....	.....	.....	.....	.....	.....
" 6th, 1896.	.....	2 100	2 250	.....	.....	.....	.....	.....	.....
" 10th, 1896.	1 900	.....	2 075	.....	.....	.....	.....	.....	.....
" 11th, 1896.	.....	.....	2 075	.....	.....	.....	.....	.....	.....
" 13th, 1896.	.....	.....	2 050	.....	.....	.....	.....	.....	.....
" 15th, 1896.	.....	.....	2 075	.....	.....	.....	.....	.....	.....
" 17th, 1896.	.....	.....	2 075	.....	.....	.....	.....	.....	.....
" 19th, 1896.	.....	.....	2 000	.....	.....	.....	.....	.....	.....
" 20th, 1896.	.....	.....	2 000	.....	.....	.....	.....	.....	.....
" 21st, 1896.	.....	.....	2 000	.....	.....	.....	.....	.....	.....
" 24th, 1896.	.....	1 750	2 000	.....	.....	.....	.....	.....	.....
" 27th, 1896.	1 775	1 800	.....	.....	2 200	.....	.....	.....	.....
" 28th, 1896.	.....	.....	.....	2 000	.....	.....	.....	.....	.....
" 29th, 1896.	1 750	.....	.....	.....	.....	.....	.....	.....	.....
Mar. 2d, 1896.	1 750	1 800	.....	2 000	.....	.....	.....	.....	.....
" 3d, 1896.	.....	.....	.....	2 000	.....	.....	.....	.....	.....
" 4th, 1896.	.....	.....	1 950	.....	.....	.....	.....	.....	.....
" 5th, 1896.	1 750	.....	.....	.....	2 200	.....	.....	.....	.....
Bands: 5 ins. center to center.									
" 5th, 1896.	1 500	170	.....	.....	1 700	1 800	1 950	.....	2 350
" 6th, 1896.	1 650	.....	.....	.....	.....	.....	.....	.....	.....
" 7th, 1896.	1 600	.....	.....	.....	.....	.....	.....	.....	.....
" 9th, 1896.	.....	.....	.....	.....	.....	.....	2 000	.....	.....
" 10th, 1896.	.....	.....	.....	.....	.....	.....	2 000	.....	.....
" 11th, 1896.	1 475	.....	.....	.....	.....	.....	1 975	.....	.....
" 12th, 1896.	1 475	.....	.....	.....	1 700	1 800	2 000	.....	.....
" 13th, 1896.	.....	.....	.....	.....	.....	.....	2 000	.....	.....
" 14th, 1896.	1 475	.....	.....	.....	.....	.....	2 000	2 150	2 300

This maximum strain can be attained only by re-cinching after the pipe has become filled and is under pressure, which is rarely, if ever, done in practice. In case the pipe is cinched up to the limit, the strain in the band may be $f\,t\,E''$ before filling. When pressure comes on, the staves will be forced outward, owing to the compression of the wood immediately under the bands, and to deflection between the bands, to which the author has duly called attention. While this motion is small it lessens materially the lateral compression of the staves, and relieves to that extent the portion of the band strain due to stave compression. The extent of relief being unknown, it would not be practicable to take cognizance of it in the determination of maximum strains, but so far as formula (a) is concerned it may be stated that as P becomes larger for any given pipe, y becomes smaller until finally leakage results.

An experiment made by the writer some three years ago may be of interest here as illustrating the dependence of stave compression on water pressure, although it was originally undertaken to determine

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the safe maximum spacing for pipe with thin staves, and to inquire into the degree to which it would be desirable or imperative to carry the rather expensive process of cinching.

A section of 10-in. pipe about 8 ft. long was built of redwood staves $1\frac{1}{16}$ ins. thick, and having $\frac{3}{8}$-in. round steel bands with cold-pressed threads. Upon test these bands showed a tensile strength at the elastic limit of from 4 100 to 5 600 lbs. and a breaking strength of from 6 900 to 7 300 lbs. The bands were thoroughly cinched, and heads were placed over the ends of the section and connected by longitudinal bolts. One of the bands was provided with the strain-measuring device described and illustrated in a previous paper by the author.* Fig. 1, Plate I, of that paper shows the device in position on the test pipe.

The first experiment was with bands spaced at 10 ins. from center to center, and lasted 35 days, during which time pressure was kept on the pipe up to 20 lbs. per square inch. The second experiment, with bands 5 ins. apart, lasted 9 days, and the pipe was subjected to pressures up to 64 lbs. per square inch.

The actual band strain is recorded in Table No. 10.

Upon the basis of band strains at the end of each experiment for various gauge readings the analysis given in Table No. 11, shows the strains in the band and in the staves.

TABLE No. 11.—STRAINS IN 10-IN. REDWOOD STAVE PIPE UNDER PRESSURE, IN POUNDS.

Gauge reading.	Band Strain.			Average lateral compression of staves per square inch.	Pressure of band on stave per lineal inch.
P.	Total measured. *S.*	From water pressure. *P f R.*	From stave compression. *f t y.*	*y.*	
	Bands: 10 ins., center to center.				
0	1 750	0	1 750	165	285
5	1 800	253	1 547	146	293
10	1 950	506	1 444	136	318
15	2 000	759	1 241	117	326
20	2 200	1 012	1 188	112	359
	Bands: 5 ins., center to center.				
0	1 475	0	1 475	277	241
20	1 700	506	1 194	224	277
30	1 800	759	1 041	196	294
40	2 000	1 012	988	186	326
50	2 150	1 265	885	166	351
60	2 300	1 518	782	147	375

NOTE.—Actual diameter, 10.13 ins. Bands were cinched very hard. Some fibers on bastard staves had snapped. Crease made by band measured from $\frac{1}{16}$ to $\frac{1}{4}$ in.

Tables Nos. 10 and 11 show some interesting facts in relation to lateral stave compression for different spacing of bands, to which ref-

* "The Astoria City Water-Works," *Transactions*, Am. Soc. C. E., Vol. xxxvii, p. 1.

erence will be made hereafter. For the present it is sufficient to note the decrease of band strains from stave compression, $f t y$, with the increase of water pressure. Mr. Henny.

The maximum band strain as expressed by formula (*b*) is greater than that required for tight pipe. When, on a pipe under pressure, the strain in a band is released by turning the nut, the compressive strain in the staves will be decreased, until finally, leakage in the seam joints results, commencing half way between the bands, where the compressive strain is least. Calling the stave compression half way between the bands y_1, and that at the bands y_2, the average stave compression y will be approximately $\frac{y_1 + y_2}{2}$, and the condition of tightness will be expressed by $y_1 > P$.

Where the spacing of the bands is not wide, and where the staves are not unusually thin (conditions referred to hereafter) y_2 will generally be but little greater than y_1, and consequently $y_1 + y_2$ will be little greater than $2\,y_1$, and, with the exceptions noted, it is believed that tightness is generally insured by the condition $y_1 + y_2 = 3\,P$ or $y = \frac{3}{2}P$. Substituting in formula (*a*) there results for the least band strain in tight pipe:

$$S = P f R + f t \frac{3}{2} P$$

which should be equal to the safe tensile strain in the band; hence

$$s = P f \left(R + \frac{3}{2} t\right)$$

So far as indentation under the band is concerned, the safe bearing power under the band should not be exceeded by the safe band strain; therefore

$$s = P f \left(R + \frac{3}{2} t\right) \leq = \left(R + t\right) e \ldots\ldots\ldots\ldots (c)$$

from which the spacing

$$f = \frac{s}{P \left(R + \frac{3}{2} t\right)} \leq = \frac{\left(R + t\right) e}{P \left(R + \frac{3}{2} t\right)} \ldots\ldots\ldots\ldots (d)$$

Some stave-pipe specifications have required that the greatest strain in the band should fall within the limit of elasticity of the steel, so that no permanent elongation may result from either excessive cinching or from subsequent swelling and pressure strains. This condition is generally complied with, under ordinary proportions of pipe design, and usually the wood in the staves will commence to fail under the cinching strain long before the elastic limit in the band is reached. In extreme cases, however, of wide spacing, large diameter,

Mr. Henny. very dry lumber, hard cinching and quick filling and putting under pressure of the pipe, the elastic limit may be temporarily passed, but the writer can discover no special harm in this, since the soaking of the wood would quickly release the excessive strain. It is for this reason also that in the above formulas no reference has been made to temporary strains and because with stave pipe built of staves and bands of ordinary proportions, with the proper kind of lumber, there is no danger that the temporary band strains will approach too closely to the breaking point.

Formula (*d*) has been used by the writer for many years to determine the spacing of pipe bands, and he sees no reason to deviate from this practice. It should be remembered, however, that the formula is based upon the assumption that the difference between the compressive strain in the wood at the band and that between the bands is not great, an assumption that is not safe when the spacing is wide or the staves are thin. The exact effect of the deflection of the staves between the bands is exceedingly difficult to determine, and as a practical solution of this question the writer has generally fixed upon some maximum spacing which practice and experiments have shown to be safe.

For staves run from 2-in. lumber the maximum spacing has generally been made 12 ins., which has been reduced, however, to 11 and 10 ins. as the diameter of the pipe increased, without a corresponding increase in the stave thickness, in order to give to the pipe the rigidity required to resist deformation and to support top loads.

For staves run from 1½-in. lumber the maximum spacing has been fixed at 10 ins., because the staves deflect more easily, and consequently there is a greater difference in the compressive strains at points between the bands and under the bands. Thus, for light heads the spacing is made closer than would result from the use of formula (*d*), nor is the reduction of spacing confined to pressures below the limit for which formula (*d*) indicates the assumed maximum spacing to be sufficient, but the reduction is graduated over higher pressures as well, up to some point, the selection of which must be largely a matter of judgment and experience.

If the reasoning on which formula (*d*) is based be correct, exception must be taken to the author's determination of the factors of safety in the bands of existing pipe lines as given in Table No. 5.

The column "Resulting factor in band, including all strains," would show different values, as the total band strain upon which it is based would be determined by the addition of strain from water pressure and strain from swelling staves, without the further addition of an assumed strain resulting from cinching. The writer does not wish to imply, however, that the values found by the author for resulting band factors are in themselves too low. While it is held that no addition should have been made for cinching strain, it is quite possible

that the swelling force E'' may at times be higher than 100 lbs., as assumed by the author. Mr. Henny.

To determine actual factors in existing pipe lines by means of formulas alone is considered impracticable, because the compression of the staves in the pipe, when saturated and under pressure, varies with the degree of cinching of the bands and the dryness of the staves at the time of construction. These are generally unknown or at least very uncertain, and even were they known, their exact effect on final stave compression would still be in doubt. It may be assumed that the author gave probable rather than either actual or minimum factors, and, if so, the writer believes that the assumption $E'' = 100$ lbs. is sufficiently high; but that, as stated, the supposed strains resulting from cinching, given in Table No. 5, should have been omitted in the determination of probable total band strains.

It is evident that any change in the total band strains would effect corresponding changes in the computed band pressures on the staves as given in the last two columns of Table No. 5.

It will be observed that formula (d), contains but one coefficient to be determined by experiment, namely "e," the resistance the staves offer to indentation by the bands.

The necessity for its determination was promptly realized by the writer, and he, so far as facilities at hand would permit, conducted some simple experiments with redwood. Rods of various diameters were laid on the flat side of dressed samples, the position of the rod being in each case perpendicular to the direction of the fiber. The test pieces were $1\frac{13}{16}$ ins. thick and $5\frac{13}{16}$ ins. wide, dressed on four sides. Edge-grain, diagonal-grain and bastard-grain samples were selected. One set had been kiln-dried and averaged 1.67 lbs. per lineal foot in weight, while another set had been submerged in water for ten days and showed an average weight of 2.10 lbs. per lineal foot.

After each sample had been placed in position and the rod had been laid upon it, a load was applied by means of a weighted lever, the weight being slowly increased and the width of the bearing measured, until the first evidences of cracking wood fibers appeared.

Table No. 12 gives the greatest load and corresponding width of bearing before damage to the wood resulted.

The writer fully realizes that wood, even of the same tree, is a very variable material, and that experiments made on single samples can have no far-reaching importance. The results are believed, however, to be not devoid of interest and to warrant some general conclusions.

It will be noted that the width of bearing surface generally approximates closely to the radius of the rod before injury to the wood results, as has been assumed by the author. It will further be observed that the direction of the annular rings has a decided influence on the bearing power, the edge grain sustaining 60 to 100% greater loads than

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TABLE No. 12.—Width of Bearing, Load per Lineal Inch and Load per Square Inch of Bearing Surface of Round Rods laid on Clear Redwood, Perpendicular to the Direction of the Fiber.

Diameter of rod, in inches.	Edge Grain.		Diagonal Grain.		Bastard Grain.		
	Dry.	Wet.	Dry.	Wet.	Dry.	Wet.	
¼	5/32	5/32	.....	3 1/32	5/32	5/32	Width of bearing, in inches.
	317	277	.....	286	179	179	Pounds per lineal inch.
	2 028	2 216	.....	2 614	1 432	1 145	" square "
⅜	6/32	7/32	6/32	6/32	5/32	5 1/32	Width of bearing, in inches.
	415	415	317	317	180	228	Pounds per lineal inch.
	2 213	1 897	1 674	1 674	1 152	1 398	" square "
½	8/32	8/32	8/32	8/32	8/32	7/32	Width of bearing, in inches.
	518	415	415	461	267	228	Pounds per lineal inch.
	2 052	1 660	1 660	1 844	1 068	1 042	" square "
⅝	10/32	10/32	10/32	9/32	7/32	11/32	Width of bearing, in inches.
	510	510	460	448	317	400	Pounds per lineal inch.
	1 632	1 632	1 472	1 792	1 127	1 172	" square "
¾	12/32	12/32	12/32	9/32	12/32	12/32	Width of bearing, in inches.
	558	558	508	460	415	415	Pound per lineal inch.
	1 785	1 785	1 354	1 635	1 106	1 106	" square "
⅞	14/32	14/32	13/32	14/32	14/32	14/32	Width of bearing, in inches.
	701	701	608	608	415	458	Pounds per lineal inch.
	1 602	1 602	1 286	1 380	948	1 046	" square "

the bastard grain, the diagonal grain giving intermediate values. This was not entirely unexpected. Redwood, like most soft woods, consists of alternate hard dark rings (summer wood) and soft light-colored rings (spring wood). In edge grain the rod is supported by the upper edges of a large number of hard rings, while in bastard grain it lies flat on one of these rings, which receives its own support from a soft ring underneath, and is easily pressed down and broken.

Another interesting fact disclosed by the experiments is that the sustaining power per square inch grows smaller as the diameter of the rod increases. An explanation of this may, perhaps, be found in the fact that the indentation from a round rod produces not only downward compression, but also a stretching of the upper fibers in which the rod lies suspended, as it were, and the supporting effect of these stretched fibers must be proportionately greater for narrow than for wide rods.

Finally, the rather curious fact is developed that wood, when becoming saturated, does not seem to lose in bearing power. The fiber when wet does not break and snap off readily, which permits more deformation without injury, and such increased deformation does not necessarily result in a greater width of actual contact.

While generally the loads were borne for only a short time, they were sometimes left on for several hours or throughout the night, yet no further marked settlement appeared to occur. The writer, therefore, concludes that, contrary to the assumption of the author, there is no great difference between the temporary and the permanent sustaining power, E' and E.

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The supporting power of saturated redwood for a $\frac{3}{4}$-in. rod was found to vary from 415 to 228 lbs. per lineal inch of rod, according to the grain of the wood, while the 10-in. pipe experiments previously noted (Table No. 11) gave a maximum of 375 lbs.; by which pressure, however, the fibers of the bastard-grain staves had been injured and the bearing increased to a width greater than the radius of the rod.

Comparing the results of the experiments with the value of E, assumed by the author, on page 38, it is found that the assumption $E = 240$ lbs. for $\frac{1}{2}$-in. bands is about correct for bastard grain. As to the safe bearing power, e, the writer believes that the experiments indicate sufficiently that the value to be given to it for the various diameters of bands should not be based on a uniform safe pressure per square inch of contact, but that it should decrease with an increase of diameter, and he has generally based the spacing on Table No. 13, deduced from the above experiments.

TABLE No. 13.—Assumed Safe Loads of Round Bands on Clear Redwood Staves.

Diameter of band, in inches.	Width of bearing, in inches.	Safe Load, in Pounds.	
		Per square inch.	Per lineal inch.
$\frac{3}{4}$	$\frac{7}{16}$	747	140
$\frac{7}{8}$	$\frac{7}{16}$	700	153
1	$\frac{3}{8}$	660	165
$1\frac{1}{8}$	$\frac{3}{8}$	640	200
$1\frac{1}{4}$	$\frac{7}{16}$	630	232
$1\frac{3}{8}$	$\frac{7}{16}$	600	262
$1\frac{1}{2}$	$\frac{7}{16}$		

If it be desired to determine the maximum strain that resistance to indentation enables the staves to transmit to the bands, values for E, equivalent to from 2 200 to 1 600 lbs. per square inch of band contact should be used.

The swelling force and the lateral compressive strength of the wood E'', ordinarily need not be considered in the determination of band spacing, for reasons previously stated (which do not apply to the design of tanks, owing to greatly different proportions and conditions). They should, however, receive careful consideration in ascertaining the limits of pressure for stave pipe and in deciding upon the methods to be followed in construction, especially in regard to the extent to which back-cinching should be carried to insure a tight pipe. The writer has made a series of experiments to determine the value of E'' for redwood and for Oregon fir, which show widely varying results, partly due, no doubt, to differences in the wood itself, and partly to the difficulty of securing an equal degree of dryness for all the samples when put under test.

Mr. Henny. The test pieces were generally from 10 to 12 ins. in length, and they were compressed in the direction of their width, which was about 5½ ins. The thickness varied from 1½ to 2½ ins. The test pieces were placed between two 1-in. iron plates, each provided with a hole corresponding to a hole bored through the sample in an exactly central position. A ⅝-in. bolt was passed through the hole, and upon the projected thread was placed the measuring device previously mentioned. The pieces had been kiln-dried and had been kept in a dry place ready for use, but it is very probable that some moisture had been reabsorbed. The bolt was usually tightened until a strain of from 4 500 to 7 000 lbs. was reached, and then the test piece was submerged in water either with the original strain left on the bolt or after the strain had been released to about 1 000 lbs. or less. The strains were read from day to day until there seemed to be no further change, when the sample was taken out; or it was put back in the water with the strain increased or released by turning the nut; or it was placed in dry air, as indicated in Tables 14 and 15. It should be stated that the maximum movement permitted by the spring of the measuring device was 0.02 in. for a 10 000-lb. load.

TABLE No. 14.—Changes of Compressive Strain in Clear Redwood when Becoming Saturated.

Number of test pieces.	Direction of grain.	General dimensions, in inches.	Pressure in wood per square inch of compressed section, in pounds.	Weight per foot B. M., in pounds.		Remarks.
				Before experiment.	After experiment.	
27	Edge.	2½ × 12	50– 77 (20) 171–187 (15)	2.14	3.04	
28	"	2½ × 12	27– 60 (17) 30– 40 (12)	2.12	3.25	
29	"	2½ × 12	197–184 (21)	2.16	3.01	
9	"	1½ × 10	491–161 (27)	2.16	3.09	
10	"	1½ × 10	44–161 (15)	2.14	3.02	
11	"	1½ × 10	471–175 (32) 31–104 (17)	2.13	3.30	Grain had slipped slightly
4	"	1½ × 12	28– 53 (18) 140–112 (12)	1.88	3.33	
8	Diagonal.	2½ × 12	24– 78 (14) 217–131 (14) 239–141 (22) 254–154 (24)	1.75	2.21	
12	"	1½ × 10	74–108 (14)	2.50	3.38	Had been cinched to 471 before release and submersion, and grain had slipped.
13	"	1½ × 10	363–144 (21)	2.47	3.29	
15	Bastard.	1½ × 10	51–116 (21)	1.64	2.25	
16	"	1½ × 10	348–191 (18)	1.75	2.79	
17	"	1½ × 10	333–167 (21)	1.71	3.29	
10-in. test pipe.	.....	1$\frac{7}{16}$ × 10	404–245 (1) 245–165 (33)			
10 " "	.....	1$\frac{7}{16}$ × 5	282–277 (9)			

Note.—Numbers in parentheses indicate duration of each experiment in days.

TABLE No. 15.—CHANGES OF COMPRESSIVE STRAIN IN CLEAR OREGON FIR WHEN BECOMING SATURATED AND WHEN DRYING.

Number of test piece.	Direction of grain.	General dimensions, in inches.	Pressure in wood per square inch of compressed section in pounds.	Weight per foot B. M., in Pounds.		Remarks.
				Before experiment.	After experiment.	
18	Edge.	1½ × 10	[511–906] (17) 306–122 (40)	2.80	3.80	Between first and second experiments test piece was placed alternately in air and water, and with each release of pressure from drying, bolt was recinched.
19	"	1½ × 10	51–183 (14) 75–122 (9)	2.77	3.72	
20	"	1½ × 10	136–163 (8) 88–156 (2) [156– 90] (20)	2.81	2.75	
2	Diagonal.	2½ × 10	7– 74 (16) 14– 34 (12)	....	.....	
6	"	1½ × 12	28– 92 (9) 274–129 (19) 291–151 (25)	....	....	Test piece had buckled, showing a deflection of 0.07 in. in the center.
21	"	1½ × 10	122– 88 (10) [88– 34] (5) [142– 34] (8) 34– 88 (13)	2.85	3.45	
22	"	1½ × 10	54– 68 (9) 156– 99 (7) 150– 99 (6)	2.67	2.82	Test piece had buckled, showing a deflection of 0.02 in. in the center.
23	"	1½ × 10	68– 75 (7)	3.01	3.27	
24	Bastard.	1½ × 10	68–238 (14) 61–171 12)	3.25	3.82	Test piece had buckled, showing a deflection of 0.06 in. in the center.
25	"	1½ × 10	284–329 (5) 92–295 (7) [295– 13] (15) [123– 34] (6)	2.99	3.01	
26	"	1½ × 10	148–269 (14)	2.97	3.79	Same as test piece 20.
18-in. test pipe	.....	1⅜ × 12	288–153 (13) 7– 94 (17) 30– 74 (19)	2.53	3.90	
18 " "	.....	1⅜ × 12	33–121 (8) 121– 97 (23)	3.10	4.20	

NOTE.—Numbers in parentheses () indicate duration of each experiment in days. Figures in brackets [] show changes of pressure in dry air; all other figures are for test piece submerged in water.

There are many interesting deductions to be made from these experiments. The writer will confine himself to the following:

Bastard grain shows a decidedly greater swelling power and compressive strength than either edge grain or diagonal grain, which renders the selection of bastard grain staves very desirable for the high pressure portions of a pipe line, affording a further advantage in offering a greater safeguard against percolation. It will also be noted that after the first swelling strain had been released, the further strain from swelling never reached the original value and generally remained considerably below it. This emphazises the necessity of hard-cinching the bands on pipes built with staves that are not perfectly dry, a condition rarely attained.

Mr. Henny. Another interesting point brought out in the tests with fir is that repeated wetting and drying will necessitate repeated screwing up of the bolt to prevent the entire release of the pressure, but that finally a point is reached when drying out may reduce the strain, but can no longer cause complete release, which is of importance rather in tank than in pipe construction.

The tests with 10-in. redwood pipe described by the writer, and those with 18-in. Oregon fir pipe, mentioned by the author, have both been included in the tables, for convenient comparison. In regard to the former some further remarks may not be out of place.

The band on the pipe with 10-in. spacing had originally been screwed up to a tension of 4 300 lbs. (stave compression per square inch, 404 lbs.; band pressure per lineal inch, 702 lbs.). This fell in one day to 2 600 lbs. (stave compression per square inch, 245 lbs.; band pressure per lineal inch, 425 lbs.), evidently as a result of a yielding of the staves in width as well as under the band. This strain, in the course of twenty-nine days, was further reduced to 1 750 lbs. (stave compression per square inch, 165 lbs.; band pressure per lineal inch, 285 lbs.) where it remained. As with water pressure on the pipe the band pressure per lineal inch increased to 375 lbs. without yielding, it is apparent that the final reduction in lateral compression of the staves was not due to a yielding of the wood under the band, but solely to lack of compressive lateral strength.

Upon the termination of the experiments on the pipe with 10-in. band spacing it might have been concluded that 165 lbs. per square inch was the permanent compressive strength of the saturated staves. Yet when the band spacing was reduced to 5 ins. the same staves sustained a compression load of 282 lbs. per square inch, which load suffered practically no reduction during the nine days that the test lasted.

Experiments, few in number, with different samples of redwood, under varying conditions, may not offer sufficient basis for scientific deductions, still the writer is strongly drawn to the conclusion that the greater compressive strength of staves in pipe with 5-in. band spacing, as compared with that of staves in pipe with 10-in. band spacing, and of the latter as compared with that of the average of straight test pieces, may well result from the side support, which the staves receive from the surrounding bands, against folding and buckling, which was frequently observed in unsupported test samples.

The economy in the use of oval bands on small pipe, as a means of securing the required bearing surface, was fully pointed out by the author. The difference in cost per pound, between oval and round bands, as quoted by the mills, is unfortunately not as trifling as was assumed, although in general the writer can agree with the author's table No. 4, giving economic proportions for pipe design. He has,

however, often used ⅞-in. round bands in preference to oval bands on pipe lines smaller than 22 ins. diameter, in case the pressures generally were light, because up to a certain pressure the same number of round bands as of oval bands will suffice, the spacing not being governed by limited bearing surface, but by possible stave deflection between bands. Up to that pressure the use of round bands will therefore be more economical, and when the higher pressure is reached, where the cost of round and oval bands would be the same, the writer prefers the use of the former, as a greater weight of steel is obtained, causing a corresponding increase in the probable life of the pipe without extra expense. If, on such a pipe line, scattered short stretches occur, where, so far as purchase-price of material is concerned, there would be economy in using oval bands, it remains an objection, from the practical point of view of a pipe builder, to make frequent changes for short distances from the one to the other kind of bands. The author's argument applies, however, with full force to pipe lines where the pressure generally is high. Mr. Henny.

The alleged disadvantage of oval bands, in exposing a greater surface to corrosive action than round bands, is more apparent than real, at least so far as the adopted sizes are concerned, since a $\frac{5}{16}$ x $\frac{7}{16}$-in. oval band has a surface exceeding that of a ⅞-in. round band by only 1.3 per cent.

An important subject in connection with stave pipe, which the author has not touched, deserves attention. Reference is made to the possibility of leakage, which, in view of the relatively large number of seam and butt joints, might well predispose those who have had no experience with it against stave pipe. The author has mentioned this matter in his paper on the Astoria Water-Works,* where a test of tightness of some three miles of 18-in. stave pipe was described. It is believed that he has lately made some additional measurements on this subject, a statement of which would undoubtedly add to the value of the paper.

JAMES D. SCHUYLER, M. Am. Soc. C. E.—This paper will be accepted by the friends and advocates of wood stave pipe as the most systematic and logical analysis of the subject that has yet been attempted, and the author is certainly entitled to the gratitude of the profession for the very clear and concise statements of the conclusions to be drawn from the lessons of experience with this class of pipe up to the present day. Mr. Schuyler.

Reference has been made to the experiments on the carrying capacity of the 30-in. line, 16.4 miles long, supplying the City of Denver, which were conducted, in 1892, under the supervision of the writer. Unfortunately, he has not at hand the notes of these experiments in detail, although they may still be found among the records of the

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 1.

Mr. Schuyler. Water Company. In brief, they consisted of a careful notation of the time of filling a measured depth in the reservoir at the lower end of the line, all outlets being carefully closed while the experiments were in progress. They extended over a period of more than twelve hours, and were checked at frequent intervals by current-meter observations, at one of the manholes, where the pipe was near the hydraulic grade line. The result of the experiments gave the value of the coefficient n in Kutter's formula as somewhat less than 0.10, a number of computations having been made of progressive measurements during the progress of the filling of the reservoir, within the time stated. These computations gave slightly varying results, for the reason that the pipe did not discharge freely at the top of the reservoir, but entered it at the bottom, and, as the water rose, the hydraulic gradient was constantly changing and lessening the velocity in the pipe. The experiment, as a determination of the capacity of a wood stave pipe *per se*, was further complicated by the fact that the total length of the conduit was not of the same material, but, in addition to the 87 580 ft. of wood pipe, there was, near the lower end, 17 145 ft. of 30-in. cast-iron pipe, as a portion of the line involved in the test, and also a cement-lined, circular tunnel, 36 ins. in diameter and 2 288 ft. in length. For these reasons, a definite determination of frictional coefficients, applicable to the wood portion of the conduit, was not possible, although the results appeared to justify the conclusion drawn in a former paper,* that the coefficient n of Kutter's formula was, in this case, as low as 0.096 for the wood pipe. In deriving this inference it was necessary to apply to the cast-iron portion such mean coefficients as experience with that class of pipe seemed to warrant, and a mean of a number of experiments published in Trautwine and Hering's translation of Ganguillet and Kutter's work was used.

Mr. Adams. Arthur L. Adams, M. Am. Soc. C. E.—The discussions and correspondence present much that will be of substantial value in reaching ultimate conclusions regarding the questions to which the paper gives rise. The new data presented, particularly so far as they relate to the determination of probable strains, are most valuable, and in some respects would seem to warrant some change in the author's assumptions, though it is believed without influencing the final results very much. Many of the objections taken to the author's statements have, on the other hand, arisen from an imperfect perusal of the paper, a failure to grasp the measure of its intended scope, and, to some extent, from a very imperfect knowledge of the subject treated.

Mr. Henny has contributed a most interesting discussion, the subject being one on which he is remarkably well qualified, by experience, thought and experiment, to speak. The tables of experimental results are interesting, and constitute a substantial addition to existing knowl-

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 144.

edge of a subject at best not much investigated. He contends with much clearness and force in favor of the use, under all conditions, of the formulas designated by the author as (*C*) and (*D*) for the determination of band spacings, instead of formulas (*A*) and (*B*), claiming that the sum of all strains transmitted to the bands, additional to that resulting from water pressure, cannot be determined by the summation of the same. The author is disposed to believe this point well taken, as set forth in the formula (*b*), but the results of further experimentation to determine the amount of swelling strain, which he submits, would seem to indicate as advisable the assumption for E'' of a figure considerably greater than the 100 lbs. per square inch suggested by the author; and since the maximum pressure suitable for stave-pipe constructions is not greater than this, while the average pressure, in its economic field of usefulness, is very much less, it would seem that the factor, $E'' t f$, would always predominate in the production of band strain rather than the $P f \frac{3}{2} t$. The formula, $S = P f (R + \frac{3}{4} t)$, gives the band strain in a pipe which is just free from seam leakage and little more. This is the minimum permissible, and not the strain probably prevailing. The author can scarcely agree with the contention that: Mr. Adams.

"When pressure comes on, the staves will be forced outward, owing to the compression of the wood immediately under the bands, * * *. While this motion is small it lessens materially the lateral compression of the staves, * * *."

Redwood shrinks transversely in drying about 4 per cent. So long as the proportions are restrained much below that originally prevailing, it must be capable of exerting pressure. An increase of even 1% in the width of the staves by swelling would cause an increase of 1.03 ins. in the diameter of a 30-in. pipe, thus necessitating the further sinking of the bands $\frac{1}{2}$ in. into the staves, if the strain from this source is to be thus materially lessened. If, therefore, either factor is to be omitted from the formulas, in deference to the first contention, it should, as previously stated, be *X*, or its assumed equivalent $P f \frac{3}{2} t$, rather than $E'' t f$. Such a modification, with some such increase in value to be assigned to E'' as the latter experiments seem to warrant, will not materially affect the results set forth in the table of probable strains prevailing in different pipe lines.

In structural work, a suitable factor of safety is a provision against failure from more or less unavoidable defects in material, from strains abnormal or indeterminable, originating from within or without the structure, and against undue termination of life from deterioration. The author does not think that a proper regard for these requirements will ever warrant the designing of a pipe along lines anticipating the straining of the materials even temporarily beyond the elastic limit; and he believes that he has already shown that, at least in pipes of any considerable size, the yielding of the wood beneath the bands cannot be counted on to afford much release of the strain from swelling timber.

Mr. Adams. The differences in bearing power of the staves, per unit of contact for bands of different diameters, as experimentally determined, are certainly worthy of careful consideration.

Regarding the degree of unavoidable leakage in stave pipe, concerning which inquiry is made, the author, in the summer of 1898, experimented on a 4½-mile section of 14-in. stave pipe designated as No. 13 in Table No. 5. The pipe is subject to a maximum pressure of about 60 ft., and an average of, perhaps, 25 or 30 ft., and is so located that the pressure is not much affected by the volume of discharge.

The rate of flow was measured by stationary weirs and hook-gauges set in masonry structures at each end of the line. The weirs are 4 ft. in length, and the rate of discharge was first reduced to a head of about 1 in. No loss was discernible. That the test might be made more delicate, and unavoidable errors better eliminated, a temporary weir only 8 ins. in length was substituted in both cases, and the flow further reduced to a head of about 1 in. Again, no loss was discernible, and the author considered himself justified in pronouncing the pipe without leakage. Each test was continued for some time after the low-stage uniform flow was attained.

Mr. O'Rourke thinks the limitations of economic usefulness of stave pipe, in reference to first cost, should be more closely defined. The author has already pointed out, page 57, that this line is not a constant, but is variable, not only for different points but even for the same point. Tables Nos. 8 and 9 are sufficiently extended to make manifest by inspection the position of this line at the points designated.

When buried in moist earth, no more serious objections than those the author has suggested can be urged against the use of thicker staves, except that it constitutes a needless increase in cost; but experience has amply demonstrated that when exposed to the action of the air, the process of wetting and drying incident to the varying rate of surface evaporation, will in a few years induce decay on the outer surfaces of thick staves.

On page 53 a long life for light-gauge riveted pipe, under favorable conditions of soil, is already conceded; but where soil destructive to steel pipe exists in such abundance as on the Pacific Coast, a note book, not to be misleading, should contain both extremes, rather than one.

There, also, will be found the author's reasons for including in the tables steel of greater thickness than the pressure alone requires.

If the statement therein made that "the exceptions are instructive, and fully vindicate the position assumed by many engineers, who, for permanent work, have recognized the necessity for a considerable thickness of metal, independent of the demand for a sufficient factor of safety against bursting, and who accord little favor to pipes of a very light gauge," is not sufficient, the author would suggest a study

of the minimum thickness adopted in most of the leading structures of this type. The following come at once to mind: Mr. Adams.

East Jersey Water Company	$\frac{1}{4}$	in.
New Rochester Works	$\frac{1}{4}$	"
Portland, Ore., Conduit	0.203	"
Coolgardie	$\frac{1}{4}$	"

The statement that the waters of the Pacific Coast States "are nearly always hard," will come as a shock to the engineers familiar with that region of granite and basalt; and likewise startling is the statement that tuberculation does not there exist. The writer has taken up cast-iron pipe in Oregon, which, after carrying water from a mountain stream—the source in the Northwest most frequently utilized—for nine years, was all so heavily tuberculated as to certainly reduce its carrying capacity 25%, while a recently completed and exhaustive examination of the works of the Los Angeles City Water Company by a commission of four engineers, of which the author was one, disclosed the fact that all pipes were tuberculated after being in service only six years. It was estimated by the commission that, on an average, the capacity of cast-iron pipe was diminished from this cause 10% in the first ten years, and thereafter at the rate of $\frac{1}{2}$ % per annum. Some of the pipes examined had been in service thirty years, and the water conveyed by them, containing about 26 grains of solid matter per gallon, chiefly carbonates of lime and magnesia, would be classed among the harder waters of the Pacific Coast.

The table of probable relative discharging capacities is not intended to be either prophetic or more exact than the present state of investigation warrants. Many modifications will doubtless be rendered necessary by the results of future experiment. The author has nowhere claimed more for the table than that it is abreast of the present knowledge, and while freely admitting the presence of apparent irregularities, and the need for much additional investigation, he has no hesitancy in affirming that the order of relative discharging efficiency of the different classes of pipes, as set forth in the paper, will not be changed. The difference will be one simply of degree, not of fact. That it is substantially abreast of present knowledge he strenuously insists; and, in addition to what has already been said relative to this matter, only wishes to refer the skeptical to that admirable review of nearly all existing data on the discharge of riveted pipes of large size, as set forth in Herschel's "115 Experiments" new and old, and to state that experiments on any conduit having a smooth interior, sheds much side light on the probable discharge of stave pipe. The admirable experiments of Messrs. Stearns and Fitz-Gerald,* on a double line of 4-ft. cast-iron pipe, tuberculated and

* *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 241.

Mr. Adams. clean, are valuable in this connection.

Mr. Holbrook says stave pipe was in use as early as 1865. Mr. J. D. Fouquet,* M. Am. Soc. C. E., says as early as 1850. J. T. Fanning,† M. Am. Soc. C. E., says he built a line 6 ft. in diameter at Manchester, N. H., in 1874. Mr. Le Conte says it has been in use eight years.

All that Mr. Patton states regarding the experience with stave pipe at Cheyenne is exactly what might be expected under conditions so manifestly unsuited for its use as there prevailed.

The author has from time to time heard that growths may occur in stave pipe, but has yet to learn of a single instance where anything of the kind has occurred, with the pipe constantly full and under pressure.

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 172.
† *Transactions*, Am. Soc. C. E., Vol. vi, p. 72.

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No. 844.

ROAD BUILDING.

By Messrs. JAMES OWEN, W. E. MCCLINTOCK, HENRY I. BUDD, N. P. LEWIS, CALVIN TOMKINS, G. A. ROULLIER, E. W. HARRISON, J. J. MCLAUGHLIN, G. W. TILLSON, CHARLES H. GRAHAM, HOWARD J. COLE, HENRY B. SEAMAN, GEORGE HILL, S. C. THOMPSON, EDWARD P. NORTH, T. MCC. LEUTZÉ, WARREN B. TRAVELL, F. G. CUDWORTH, T. H. MCCANN, M. D. BURKE, H. CONROW, L. J. LECONTE, W. C. FOSTER and JAMES OWEN.

AN INFORMAL DISCUSSION PRESENTED AT THE MEETINGS OF THE SOCIETY ON SEPTEMBER 21ST AND NOVEMBER 2D, 1898.

JAMES OWEN, M. Am. Soc. C. E.—The question of road construction in latter years has assumed a somewhat different aspect from that which it bore some five or six years ago when the subject was rather thoroughly discussed by the members of this Society. Mr. Owen.

Attention should be called to what may be termed the competition in the road question as evidenced by the lowering of the prices for ordinary street pavements. The speaker remembers the time when the cost of building a telford pavement was about $2 per yard. To-day asphalt pavements are being built at a cost of about $1.38 per yard, and the engineer, in selecting a pavement,—if he has any discretion in selecting—might suggest to his clients that it might be better in many places to choose a permanent street pavement, at the lower price which it now costs, than an ordinary macadam road. This idea is advanced because it seems to solve a good many prob-

Mr. Owen. lems. The construction of macadam or telford roads extends through different territories and different kinds of population, and can be divided into what may be called mural, suburban and rural. Now, each of these kinds of road requires, to a certain extent, a special treatment, as far as construction for the travel and the available funds with which to construct it are concerned; and it seems to the speaker that it would probably be better to confine the discussion to rural roads, because in the future more permanent and less destructible pavements will be substituted in a great many cases, where macadam would now be built.

There are two points to which attention should now be called: *First*, the material; and, *second*, the handling and laying of the material. Some years ago the speaker read a paper* before this Society, and stated that he did not believe in thin pavements, or in anything but a telford pavement, as far as macadam was concerned, in ordinary circumstances. His mind has not very materially changed in this respect, and he still adheres to the principle, both in theory and practice. Mr. Leutzé states that he put down an 8-in. macadam road. The speaker hardly thinks this is a wise practice, and believes that a road would be cheaper, very serviceable, and would give better results if 5 ins. of telford and 3 ins. of macadam were used instead of 8 ins. of macadam only. In the first place, in most parts of the country there is available local stone, which is not perishable, which can be used for a foundation, leaving only the final finish of 3 ins. to be provided from other sources. In the speaker's opinion, it is cheaper, in the majority of cases, to build an 8-in. or a 10-in. telford than an 8-in. macadam, and the final results are much better.

The next problem is the question of the selection of stone. It is conceded, on all hands, that trap rock is the *sine qua non* for macadam and telford pavements. There are different kinds of trap rock and they work differently in treatment, but it probably will be agreed by all that a trap rock pavement, if it can be built as cheaply, or even at a slightly greater cost than other stone, is the best. Certain country roads built with granite of rather a soft quality, have been examined by the speaker, and, for the use to which they are put, they are quite successful. A well-selected granite, in one or two respects, is preferable to trap; it is more friable, and, though it wears out a little faster, it does not break up in dry weather, and where it can be had with a short haul there is no specific objection to it. Of course, granite containing too much mica will not be successful.

In a limestone country, if properly handled, properly laid and judiciously finished, successful results can be had with a good limestone road. Its defects are known. It grinds out more easily than either trap or granite. It is dusty. It is also known that a limestone road

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 608.

is much preferable to any other construction, except either trap or granite. Mr. Owen.

Then there is the local use of the shales and gravels without treatment. Construction of a permanent gravel road on an aboriginal granite foundation can hardly be classified as an engineering improvement in the way of roads. Yet the proper use of gravel, carefully handled, thoroughly rolled and well selected, can make a very satisfactory road for Mr. North's suggestion of nine months in the year, but not for more than the nine months. There are also the shales, which can be treated and make really a better road for nine months in the year than limestone.

In naming these materials, nearly all the material for roads in this section of the country has been exhausted. In the southern States there are other materials. In Tennessee there is an excellent natural material that makes fair traveling roads; and, of course, in the South, roads are not so apt to be broken up by frost, and all the care necessary is to keep the water away and keep them from washing out.

In regard to the size of the stone, the speaker some years ago laid down as a principle that all stone for roads should pass through a $1\frac{3}{4}$ or $1\frac{1}{2}$-in. ring. He has modified this view, having found that the locality of the roads should be considered, and now he classifies the sizes of the stone according to the character of the travel. While a very satisfactory driving road can be secured from $1\frac{1}{2}$-in. stone, for ordinary country travel and for nice driving travel in a suburban community, the same cannot be said where travel is heavily congested; and, of late years in his practice, the size has been increased to from 2 to 3 ins., not only on the score of economy, but to secure better results and a better and more uniform surface; while, by the abandonment of the ideal $1\frac{1}{2}$-in. stone, the smooth surface is sacrificed, and a generally better average result is obtained by the use of the larger stone. It does not do to have the sizes mixed. If a road is built with $2\frac{1}{2}$-in. or 3-in. stone, it should be all $2\frac{1}{2}$-in. or 3-in.—no dust, no small stone intermixed. If on a road there is a patch of 2-in. stone, and then beyond that a little patch of $1\frac{1}{2}$-in., the latter will wear out more quickly than the former, and in a year or two there will be an uneven surface, which should be avoided. The speaker wishes here to emphasize this point to those who manage the industry, in which a large amount of capital is invested, of breaking stone for the market. The difficulty is that in the arrangement of the crackers and the adjustment of the screens sufficient consideration is not given to separating the uniform sizes of stone to be delivered. The men who handle the stone crackers and stone breakers are too apt to dump it in a heap, and if 2-in. stone is asked for, the answer is: "We will give you a little finer, but it doesn't hurt." It does hurt. The use of anything but uniform sizes is a material drawback to the

Mr. Owen. final success of the road. If engineers will insist on that point, the result on the wear and tear of the roads will be much more satisfactory, not only to the engineer, but to the community.

The question of the top finishing of the road is a matter about which the speaker is not particularly solicitous. The top finishing of a road is an accessory. If on a heavily traveled road a coating of dust is placed, it probably will last from one to three months, after which the travel is down on the hard wearing surface below. In roads that are constructed for fancy driving, dust is a very pleasant accessory, and is desirable on that account; on a rarely traveled country road it is very desirable because it prevents the breaking up of the stone surface below in dry weather, and, to a certain extent, prevents wash in very wet weather. Generally speaking, however, the question of the dust is not of importance. A suggestion, however, has been made that a mixture of the trap rock as the ideal stone and the limestone, which has a better cementing power, might be advisable.

Another point of which mention might be made is a much controverted question—the packing, that is, the injection of extraneous matter with the stone for the purpose of binding. This packing is nothing but a temporary expedient to get the road down to its permanent condition, and, as such, is a detail of construction and not a permanent factor in the wear and tear of the road. A great deal of controversy has arisen as to whether gravel, sand, clay or loam is the best material for packing. In the speaker's experience almost any available material, carefully selected, will do the work required of it. Gravel or sand may be used, but the speaker does not recommend either. Clay should be used very sparingly. Ordinary earth, or loam, as it is called here, is about the most efficacious material to use for the purpose of getting a road down speedily. Clay is treacherous. The speaker has known a piece of road to be utterly ruined by an overdose of clay in its construction. It breaks the road up continuously, and the damage is never rectified until another coating is put on to eliminate the original error. It is merely a question of judgment on the part of the engineer as to how he will use packing, and in using that judgment the element of weather is the most important consideration, outside of the question of the character of the stone itself. A hard Orange Mountain stone needs a great deal more packing to get it down than does the Bergen trap stone, and a Bergen trap stone needs a great deal more than granite. The granite wants very little. The limestone wants hardly any at all. The shale furnishes its own packing. But, going back to the point mentioned, the weather is an important consideration in this item of of packing—not only the weather as it is to-day, but the probable weather conditions that may exist in the two or three months before the road is finally consolidated. If a period of dry weather can be

anticipated, it is reasonable and desirable that a great deal more packing should be used than if the road is built late in the fall when there is a prospect of wet weather ahead. In the latter case packing should be used very sparingly. It is entirely a matter of experience and judgment. If care is used in the selection and application of the packing, a great deal of the trouble that engineers find in building roads throughout the country, and the criticisms made that the packing should not be there, does not belong there, and that it is a fraud on the part of the contractor to put it there, will be eliminated. Mr. Owen.

The speaker has known whole communities torn to pieces on the question of packing, and governing bodies indicted on the question, simply because the public mind was not educated as to what was proper practice at that place and time.

There is one other point which bothers all who are building roads, namely, the interjection of a trolley track in the middle of a telford pavement, and it is very important that it should be discussed. The conditions of maintenance of a pavement with a trolley track in it are entirely different from the conditions of the ordinary road on which the travel is uniformly spread over the entire surface. The laying down of a rail, and the laying of a pavement on each side of the rail, causes a distinct and well-defined line of travel on each side of the track. The trouble is that instead of the pavement wearing uniformly over its surface, that *bête noire* of all road engineers—the rut—is started. This question of ruts is one of the most important with which we have to deal. In the last two or three years the speaker's practice has been changed altogether. Instead of 2½-in. stone he uses 3-in. stone, and each side of the track is made a separate roadway of itself with its own individual crown. The experiment of putting a coating of gravel on the top pavement, at first as a matter of economy, has also been tried. Experience shows that the increase of wear and tear and the cost of maintaining a similar area of roadway on each side of a trolley track is about twice what it would be with the ordinary condition of a single macadam or telford road. In Providence, which is practically a macadam city, the same problem has arisen. It is honeycombed with trolley tracks like all American cities, and in maintaining the roads larger stone is being used. The City Engineer of Providence in 1897 said, in reply to a question, that the final result arrived at there was to rip them all out. It may be that that will be the only solution. Up to now there has been no satisfactory solution of the maintenance of that kind of road-bed.

W. E. McCLINTOCK, M. Am. Soc. C. E.—In Massachusetts, in 1892, a temporary Commission was appointed to investigate the whole topic of road building in that State. In 1893 the Commission was made permanent. In 1894 it was given an appropriation of $300 000; in 1895 it was given $500 000; in 1896, $600 000; in 1897, $800 000, and in Mr. McClintock

Mr. McClintock

1898, $400 000, all of which was to be expended upon roads petitioned for by the Selectmen of the different towns or the Commissioners of the different counties.

A comprehensive scheme was first laid out, which included main roads throughout the length and breadth of the Commonwealth, amounting in the aggregate to about 2 000 miles, and an effort was made to direct the petitioners, to a certain extent, so that the roads petitioned for might form parts of a completed whole. Up to the present time there have been received by the State Commission 460 different petitions, covering 1 094 miles of road, and of these petitioned roads there have been laid out and constructed 223 miles. Each mile of road which has been constructed forms a part of one of the main roads before mentioned. The Commission at the beginning of its work selected the most difficult roads for treatment, the object of this action being two-fold: first, to better the entire road by removing the heaviest obstructions to travel, and, second, to show a reduced average cost rather than a uniform or increasing cost. In taking the cost of work, as it has been carried out in Massachusetts, this fact must always be borne in mind. The average cost of the work done by the Commission has been $10 300 a standard mile, of broken stone road 15 ft. wide, with gravel shoulders 3 ft. wide on each side of the stone, and including the incidental items of grading, draining, foundation, culverts, bridges, fencing, engineering, etc.

When road engineers fail to agree, the probabilities are that their conditions are entirely different. In describing the work which has been done by the Commission in Massachusetts, perhaps it might be well to first outline the conditions which are to be found there.

The topography of the State may be generally described as follows: Cape Cod, a narrow strip of land, extends from 60 to 80 miles into the sea, and is the eastern part of the State. In the main it is made up of sand, with a high moraine ridge, extending lengthwise through the center. This moraine is made up mostly of a hard, boulder till, with large masses of schist, slate, and granite, either embedded in the till or scattered over the surface. The south side of the moraine is of a sandy nature and devoid of stone. The north side of the moraine is well supplied with field stone, mostly granitic, in sizes varying from a few inches to many feet in diameter. There are a few available banks of clay. Back of the Cape Cod district is the main easterly shore belt, about 10 to 12 miles wide, which is of volcanic origin, abounding in trap, felsite, granite and conglomerate, any one of which stones makes good road material, while some of them are of the best quality for this purpose. Then, working westerly, there is a broad area known as the Worcester Plateau, mostly drift. The native rock in this section is either schist, slate or gneiss, none of which can be advantageously used in the construction of roads. Westerly of the Worcester Plateau

comes the Connecticut Valley, with a long ridge of high hills, made up almost entirely of trap rock, beginning near the northern end of the State, following the river across the State and continuing into and across Connecticut. Westward of the Connecticut Valley is another level area in which is to be found a poor grade of field stone, but no good ledges. Then comes the Berkshire Hills district, which is practically devoid of good road material, the main Berkshire Hills being composed of schistose and lime rocks, with small quantities of quartzite. Mr. McClintock

Large crushing plants have been set up at various points on the lines of railroads, and these supply broken trap rock to different parts of the Commonwealth. The stone thus delivered varies in price from $1 to $1.40 per ton on the car at its destination. All the larger communities can be supplied economically in this manner, but when the road to be built lies two or more miles from the delivery point, the increased cost of haulage over the highway makes it necessary to abandon the trap and use the native rock, even though it be of poor quality. Quartzite has not given good results, as it lacks binding properties. Schist or slate has not been used. Lime rock has been used in one road, where it gives a good bond, but wears rapidly. Mixed field stone has a good bond, but wears so as to leave the harder stone projecting above the surface. Better results can be obtained if the hard stones be picked out and broken by themselves for use in the wearing course. Some of the harder granitic rocks are difficult to roll, but when thoroughly rolled make a good road. Gneiss and the softer granitic rocks have not been used.

In almost all these territories the country is undulating, with hills varying from 100 ft. to 2 500 ft. high at points traversed by roads, so that the grade problem is an important one, and the cost of grading must be taken into account. The Berkshire Hills district, or the western part of the State, abounds in clay. Over this clay the old roads have been built, and during certain seasons they are practically impassable. The roads of this section, as well as like roads in other parts of the Commonwealth, have been treated specially as clay roads, and the greatest possible care has been exercised to secure good foundations and perfect drainage. In laying out the work, and at the very beginning, the subject of cost was to a certain extent ignored, the object being to get the best road possible, and one which would require the least annual outlay for maintenance. The engineer was instructed to prepare and bring in a plan for as long a road as could be built for a certain stipulated amount of money, say $5 000, $10 000 or $20 000.

His instructions were that a grade of 5% should not be exceeded unless by special vote of the Board. The laws granting damages are so liberally interpreted by all juries in the State, that it has been found necessary to protect every embankment, having a height of say

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2 ft. or more, by a substantial guard rail, and, therefore, this item must be considered in studying the cost of the construction.

The strictest attention has been paid to drainage. Where the road passes over a wet or non-porous soil of any kind, it has been drained in the most substantial manner by placing one drain on the upper side of a scarp road, and on both sides if the road passes through a cut or over a level bottom. These drains have all been given a pitch of not less than 3 ins. per 100 ft. The trenches have been excavated $2\frac{1}{2}$ ft. below the subgrade, the depth increasing on level stretches to give the required pitch. From 2 to 3 ins. of fine, porous gravel was spread over the bottom of each trench after it was excavated. Vitrified clay pipes with bell and spigot joints were placed on this gravel, with the joints open; and around and for 6 ins. over the top was placed gravel free from sand, and which had passed through a $\frac{1}{2}$-in. mesh. Then the remainder of the trench, up to the subgrade, was filled with coarser stones, varying from 1 in. to 3 ins. in their largest diameter. All drains have been carried to an outlet, either to a culvert, to a natural water course, or to a storm sewer.

In passing over clay or a non-porous soil a telford road has been built by using 2 to 3 ins. of gravel, 6 ins. of coarse stone foundation and 4 ins. of broken stone covering. The subgrade was first shaped to the proposed cross-section and grade, carefully rolled and then covered with from 2 to 3 ins. of a porous gravel. On this gravel was placed the telford foundation 6 ins. in depth, laid in the ordinary manner with the large edges of the stones downward, the longest diameter across the road, and each stone self-supporting, not leaning one against the other. The interstices between these large stones were filled carefully with small wedge-shaped stones driven in by maul. The tops were broken off to an even surface. Then the whole surface was evened up with the tailings from the crusher or with other angular stone, and then rolled with a steam roller until it came to a permanent and unyielding foundation. This makes up what is designated as the telford foundation, or the telford road without the top of broken stone, and it has been used, as has been said, on all roads with a non-porous foundation, where it was cheaper to use the coarse stone than it would have been to replace it with broken stone. In many cases better results have been obtained by using broken stone from top to bottom of the road than by using the larger telford stone at the bottom. In placing the broken stone on the road, if it was a telford foundation, it was usually spread to give 5 ins. in the center and 4 ins. on the sides, after a thorough rolling with a heavy steam roller. Without the use of the telford foundation, and over ordinary soil, the broken stone was spread to give a depth of 6 ins. in the center and 5 ins. on the sides after rolling. On a doubtful soil, which could hardly be called a clay or a wet soil, a layer of porous gravel about 2 ins. thick was spread

on the soil before placing the broken stone, the subgrade being first rolled, then covered with the porous gravel, then with the broken stone. Mr. McClintock

All the broken stone was screened into three different sizes by the use of screens with meshes of 2½ ins., 1½ ins. and ½ in. in diameter, so that, when broken stone is referred to in any of these roads, it means stone separated in this manner into 2½-in., 1½-in. and ½-in. stone, or screenings. These have been laid on in courses with the ½-in. size, or the screenings, on top in all cases. On a country road and on all roads where the traffic is fairly light, the coarser or 2½-in. stone has been placed at the bottom, rolled thoroughly, and the surface brought to the exact theoretical distance below the top of the proposed surface with stone of the same size. This has ordinarily been placed to give 4 ins. in depth after being rolled. The 1½-in. stone has been placed on this so as to give 2 ins. depth over the whole surface after rolling, and both of these courses have been rolled thoroughly and evened up with the same sizes of stone used in the respective courses. Ordinarily, no water was applied, up to this period. Then the surface was covered with the screenings, or that part of the broken stone which had passed through the ½-in. mesh. These were thoroughly watered before being rolled ; in fact, they were drenched, so that the water passed out at the sides of the road. Then they were rolled until the finer dust in the screenings flushed to the surface, and the road was opened to traffic and kept open.

There are some roads which may be injured by the application of water even at this point. On a sandy loam road, a loamy road or a clay road over which the telford has been laid, the application of large quantities of water at this time, when the whole mass of stone is porous and open, is likely to cause puddling of the underlying soil or the subgrade, and the settlement of the larger stone of the telford or the smaller stone of the macadam; so that after a while the section of the road will become misshaped and the stone will begin to wave in front of the roller. Then, of course, the rolling process must be discontinued sufficiently long to let the road dry out, and when it is again resumed, an excessive use of water must be avoided.

In the selection of the stone, due consideration is given to its hardness and toughness. Generally speaking, trap rock is the best which can be used, although the traps which are available vary greatly in their value. The lowest grade of trap used in the Massachusetts work has a specific gravity of about 2.7, while the highest grade has a specific gravity of 3.07 or 3.08, making a difference in the weight of a little over 10%, and in the cost of fully $600 a mile under the conditions which existed on some of the roads built. The treatment of these different grades of trap is very material. The lower grade, or the rock of lighter specific gravity, will roll down very easily under the steam roller, while the higher grade, or the heavier rock, breaks

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off on the edges more or less, and in certain cases it does not compact under the action of the roller without special treatment. This special treatment may consist of the application of a little water before the screenings are laid on. It may require a small sprinkling of sand or stone screenings, or a combination of the screenings or sand and water may be necessary. Either one of these, ordinarily, will cause the stone to knit together and permit the roller to complete its work.

There have been conditions where the speaker has felt that the trap rock apparently failed to give the best results, as the breaking of the bond allowed the stone to work loose and stand on the surface. All road builders know that one of the greatest causes of wear on a country highway is the weather, and on some roads the speaker estimates that fully 90% of the loss results from the action of the weather rather than from the traffic. The binding course is blown off or is washed off, and with the harder trap rocks a sufficient quantity of the fine dust is not produced under the wheels of traffic to supply the loss caused by the weather. Under these conditions there is sure to be a loosening of the bond, and a surface on which the stone is more or less exposed. This has been one of the most serious difficulties which the Commission has encountered. There has been a great temptation at times to try some other binding material than the screenings, but up to the present time, on the 200 miles of road which have been built, nothing but the stone screenings has been used during the original construction. On certain country roads, however, when there has been no rain for a long period of time and the rocks have appeared on the roadway, a very thin sprinkling of coarse sharp sand has been used over a narrow strip in the center of the road; that is, theoretically a thin coating of sand. The practical difficulty in the use of the sand, however, is to employ intelligent men to carry on the work by the use of a small quantity only, the tendency being to use too much. For example, in visiting one of the roads a short time since, the speaker found that the whole surface had been covered with $2\frac{1}{2}$ ins., more or less, not of coarse sand, but of fine sand. Unquestionably that would prevent raveling, but not in a manner to give the best results, as the road was unsightly and yielding under the wheel. The Commissioners think that the mere sprinkling of sharp clean sand will accomplish the desired purpose.

To prevent raveling out there are two other methods which have been tried, both of them having been more or less effective. One is to cover the top of the course of stone with a very thin coating of what is termed blue gravel before spreading the screenings. This blue gravel is a natural broken stone of angular fragments. It is bound together with the finer stone dust, and a small, but varying, percentage of clay. The finer fragments of angular stone which will pass a $\frac{1}{4}$-in. mesh, mixed with the clay and the stone dust, make an excel-

lent binder when spread on the top course of stone in a thin layer (not over ¼ in. thick) and wet, then covered with the screenings and the whole mass rolled. The blue gravel and screenings together make a very hard and elastic surface, which has been found to stand both wet and dry weather perfectly well and which does not allow the surface to ravel. Mr. McClintock

There is one point in connection with the sizing up of the stone to which attention might be called, viz.: On a country road the 1½-in stone constitutes the surface coat, with the coarse or 2½-in. stone at the bottom. Where the road passes through a village or where it is a heavy-traffic road this order is reversed by placing the 1½-in. stone at the bottom and the 2½-in. stone at the top. By this method the stones are separated into sizes, and the proper sizes selected for particular uses. Some such method as that just described is made necessary by the fact that in the ordinary process of crushing the stone, there is sure to be a certain percentage of each of these varying sizes. The speaker thinks that in the crushing of ordinary trap rock about 13% of the product is ½-in. stone and finer, and about 61 or 62% of the product is 2½-in. stone, the remainder being the 1½-in. stone. By putting the stone on as has been described, viz., about 4 ins. of the 2½-in. stone after the rolling, and 2 ins. of the 1½-in. stone after the rolling, and about ¼ in. to 1 in. of the screenings, the entire product of the crusher will be used.

Fully 50% of the Massachusetts roads have been built of native rock other than trap, the greater part of it being field stone of a mixed quality, such as granite, quartzite, conglomerate, felsite, and a small percentage of the softer rocks. These native rocks have been mostly broken up at a certain stipulated price per ton, varying from 20 to 50 cents, by means of small portable crushers which have been set up by different contractors on the line of work, so that under general conditions it was absolutely essential either to use the whole product of the crusher or else pay for a certain amount of waste.

In the maintenance of the roads, up to the present time, no regular system has been developed. The best effort possible has been made in every case to keep the road in good repair. The principle of perpetual care has been adopted, keeping the roads up to the standard without letting them go to pieces, and they have been repaired entirely by the use of the screenings with possibly the slight sprinkling of sand, which has already been mentioned, to prevent the raveling out.

The cost of maintenance cannot be given at the present time. The cost of the work which was mentioned as being $10 300 per standard mile is distributed over various items, and the grading item alone, that is, the necessary cut and fill to bring the road to a maximum 5% grade, has averaged $1 250 a mile. The outlay for grading on some roads was practically nothing, and on others it has amounted to $12 000 and even $15 000 a mile where it was a mountain road, partly in rock

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and partly in clay, with heavy retaining walls on either side, numerous culverts to remove the surface water from the steep slopes, paved gutters, etc., etc. On these mountain roads, unless the water be removed quickly from the side of the road, great damage might result from a single storm, and the culverts have been built at short intervals to prevent large and high velocities. The sides of the road have been finished out to a width of 24 or 26 ft. Old tree stumps and roots have been taken out, large and loose rocks have been removed from the roadside, and generally from fence to fence the location has been left in good shape, the center constituting a hard, smooth roadway for traffic and pleasure, with the sides pleasant to look upon.

Mr. Budd.

HENRY I. BUDD, Esq.*—In the year 1891 a road law was passed in New Jersey granting State aid to the amount of one-third of the cost of the roads. The proceedings under that law are as follows: Those wishing improved roads petition to the freeholders for them, and, for the peculiar benefits conferred, agree to pay one-tenth of the cost. Then, if the freeholders of the county accept the petition, they order a survey to be made and specifications prepared. If the State Commissioner, to whom these specifications are forwarded, approves them, and the road is built, one-third of the cost is paid by the State, one-tenth by the petitioners, that is, the residents along the line of the road, and the balance, about 57%, by the county. Under that law, in 1891, $20 000 was appropriated, but not a dollar was called for, the opposition to the law being very great, especially on the part of farmers, who claimed that their taxes would be unduly increased. In 1892, $75 000 was appropriated, and only about $21 000 called for. In 1893, $75 000 was appropriated, and about $71 000 called for. In 1894, $75 000 was appropriated, and nearly the whole amount was called for

A mandamus clause in the law compelled freeholders to build roads when petitioned for, and when building began there came from different parts of the State a cry that it was tyrannical and oppressive. Strong petitions and many farmers opposed this clause before the Legislature, and it was eliminated; but the object lesson had been given in different parts of the State, and from that time applications became numerous, even in counties which had opposed the law and opposed the mandamus clause. In 1895, 1896 and 1897, $100 000 was each year appropriated and spent. In the last year even more was spent. Work is being done under a $100 000 appropriation for 1898. The State aid of $100 000 means an expenditure of about $350-000 each year, taking the State, county and individual contributions, which all hinge upon the State law and the acceptance of roads petitioned for by the Commissioner. Under that law about 300 miles have been built and about 300 miles more are petitioned for. Under the first working of the law the roads were rather expensive, as the most

* State Road Commissioner of New Jersey.

economical way to construct them had yet to be learned, and contractors had nearly their own way, which resulted in a cost of from $7 000 to $10 000 per mile. After a while stone-crushing plants began to multiply, and a great deal of competition resulted. It was learned that thinner roads were sufficient to bear the traffic, and the cost was reduced to from $6 000 to $7 000 a mile. In 1897 they cost about $5 000 a mile, and in 1898 about $4 000, and two roads 10 ft. wide and 8 ins. deep were built for $3 000 per mile. Omitting a few gravel roads from the 90 miles now being constructed, the cost will be about $4 500 a mile. At first, from 30 to 40 miles of road per year were improved; in 1897, 70 miles. In 1898, about 90 miles are under construction. In Essex County telford roads are the rule, but in other counties pure macadam construction leads. In the former there are large quantities of stone in fences which the farmers are glad to get rid of, and which make an excellent telford foundation. Through this county trap rock ridges are so common that crushers can be established along the line of each road, and thus the road can be covered cheaply with broken stone. It makes little difference in cost whether the roads are 3, 4 or 6 ins. in thickness. Mr. Budd.

In the southern part of the State 8-in. macadam is the leading depth, there being only one 6-in. road. In the country the roads are built 10 and 12 ft. wide; in the towns, about 16 ft. wide. These widths have been fixed as the limit for State aid; then, if the citizens or the municipalities wish the whole street covered, they do the balance at their own expense, and the extra paving does not enter into the State's calculation of cost. These roads are costing all the way from 23 cents to 70 cents per square yard, and where telford is used, in bad places, the cost reaches 73 cents. The average cost of these roads in 1897 ranged from 50 to 60 cents per square yard.

In the upper part of the State, in Morris and Passaic Counties, roads are built only 4 and 6 ins. deep; consequently their cost, where rock is mined along the road, ranges from 20 to 45 cents per square yard. Trap is the leading rock for construction. Commencing at the Palisades, the trap rock formation, 20 miles wide and 70 miles long, extends the whole width of the State, jutting out at convenient places in each section. The railroads run north and south, east and west through these trap ridges, and consequently the trap can be mined and easily transported to any portion of the State. The southern portion of the State is a vast plain; north of and around Trenton, a glacial drift; south of Trenton, alluvial deposit, once the bed and shore of the ocean, with no stone in the elevations or on the surface; consequently, all stone used has to be imported, and the railroads running through the trap ridges bring the stone to the southern counties of the State at a price almost as low as when transported by rail further north. The railroads seem to have a common price for

Mr. Budd. carrying stone, there being not more than 10 or 15 cents a ton difference between long and short distances, say 20 miles and 70 miles from the trap ridges. The southern and eastern portion of the State is largely a sandy plain, with occasional gravel deposits. In this portion gravel roads have been adopted with good results. Experience here shows that gravel laid upon sand makes a better road than when placed on a hard bottom. The average haul does not exceed a quarter of a mile, so that the cartage is not an important item of the cost. The cost of roads 14 to 16 ft. wide, with 8 ins. of compacted gravel, is from $1 000 to $1 500 a mile. There is now one gravel road 34 miles in length, forming, with a stone road commencing at Camden, a complete highway 64 miles long between Camden and Atlantic City, on which carriages or bicycles can travel easily and rapidly. As many as 3 000 bicycles a day pass over it, during the summer season.

The speaker's plan has been to create, as rapidly as possible, continuous highways. When the law first went into operation a little piece of road was applied for here and another there, with no connection between them; and, consequently, there was no continuity. A system of continuous avenues has now been adopted, and is very nearly completed from Atlantic City in the southern portion of the State to Jersey City, and from Jersey City to the extreme western boundaries of Morris and Passaic counties. A great many lateral roads leading into these avenues and to different county seats and important towns have also been improved. In the western section of the State another system is being built which is almost parallel with the first, so that in a few years there will be two continuous lines north and south through the State, besides many lateral roads leading into them from the east and west.

The State Aid Act at first was extremely unpopular with the farmers, and if it had not been for the wheelmen the law would probably have been a failure. The wheelmen persuaded the Legislature, that it was to the interest of the State to vote for good roads; the farmers were very strong in opposition. Now, the wheelmen are not using much effort, and the farmers are the pleaders and workers. In some parts of New Jersey a man cannot now be elected to the Legislature unless he is an advocate of these roads. It seems to be a matter of history that farmers are the earliest opponents of road improvements, and later the most ardent advocates. The speaker had been before committees of the New York Legislature, when the committee rooms were crowded with hundreds of farmers protesting that the contemplated law would ruin them, although it was developed that the towns paid 88% of the State taxes. It was also shown there that under the law some of the larger farms would not be taxed over $4 or $5 a year.

The reason that farmers in New Jersey are becoming warm advocates of the law is shown by the following example. Camden and Glouces-

ter counties are two great farming districts, in which immense amounts of vegetables and fruits are raised and hauled to the Philadelphia markets. Before the advent of stone roads, the farmers had to drag their produce from 10 to 15 miles through heavy sand. It was with difficulty that 30 to 40 baskets of truck could be taken in one load. The size of the wagons has been increased, and they now carry from 130 to 160 baskets. Formerly a great deal of produce was shipped by small boats on the numerous creeks which intersect that section of the country and are tributary to the Delaware River; the freights, cartage, etc., amounting to from 6 to 8 cents per basket. The farmer now carts his own produce to market, and saves from $10 to $12 a day. If five loads a week are carted, the saving is from $50 to $60, and, in addition, the produce is delivered to the customer in a fresher condition than when sent by boat, where three handlings are necessary. It can be readily seen that under these conditions a farming community becomes an ardent advocate of good roads. Mr. Budd.

Gneiss and granite are the leading rocks in the northwestern portion of the State, and here the practice is to use this native rock instead of trap, thus avoiding the cost of transporting the latter. These roads are consequently very cheap, and while they do not last as well as those made of trap, they can be frequently resurfaced at small cost.

N. P. Lewis, M. Am. Soc. C. E.—The speaker will simply attempt to outline the most important points in road construction. Mr. Lewis.

The idea that hard, smooth roadways are essential in the streets of large towns, but are a luxury beyond reach in rural districts, while it has been persistent, is gradually giving place to the conviction that all roads can as well be good as bad, and that the money used in improving them is profitably expended.

Many efforts have been made to express in dollars and cents the cost of bad roads, and the figures deduced have been astounding.

In Virginia, for instance (which, it must be admitted, is not famous for good roads), it is stated that the interest on depreciation of land, the additional cost and time lost in hauling, and the annual depreciation of vehicles, horses and mules, amount to $4 275 000, which is nearly $2 500 000 more than the total tax collected in the State.

The Secretary of the National Farmers' Congress has estimated that the needless cost of moving farm products in the United States,—*i. e.*, the cost beyond what it should be.—is $600 000 000 a year. Professor Latta, of Purdue University, basing his estimates on the conditions in Indiana, concludes that the difference between good and bad roads is 78 cents per acre of farm land, or $500 000 000 for the entire United States.

The fact that poor roads are the source of great expense has been so well demonstrated by good-roads leagues and other kindred associations that it need not be enlarged upon. Some States or counties

Mr. Lewis. have, by legislation, taken the matter up, and are transforming their "streaks of mud or dust" into hard, smooth roads over which far greater loads can be hauled with much less power. An enthusiasm on the subject has developed that will soon break down indifference and prejudice, and result in the permanent betterment of the roads. These attempts at improvement have sometimes been very crude. In Wisconsin, shingle sawdust, 8 ins. or 10 ins. deep, and covered with a few inches of sand, has given good results for four or five years. In prairie or alluvial districts where gravel or other coarse material is not easily available, and where stone is almost unknown, a few inches of brush or straw covered with earth has given good drainage and fair roads. Gravel mixed with a little clay has made excellent roads in localities where such material is readily obtained.

This discussion, however, should be confined to road building, not road mending, as the operations referred to may more properly be described, and road building is scarcely the application of makeshift methods, but the permanent construction of a road according to the best practice. Not that the same kind of road should be built under all conditions, but in this, as in other engineering work, the construction should be governed by the use to which it will be put, and the resources which are available to pay for it. Stone roads only can be considered permanent. The building of such roads can begin before the making of a contract for the actual work. The ultimate improvement by macadamizing all important roads should be kept in mind, and such work as is done from season to season should be determined accordingly. It is assumed, of course, that all roads have some kind of supervision, and that there is a competent officer, whether County Engineer, Superintendent or Commissioner, who is charged with the present care and planning for future improvements of the highways. (The speaker is aware that this is a bold assumption.) The dirt roads should, therefore, be well drained, both by surface ditching and under-draining where the soil is heavy. This will greatly improve the dirt road, and is an essential of permanent work. Hills can be cut down, and low places raised, gradients being reduced, and in many cases alignment can be improved before a permanent improvement is authorized; but it should be kept always in view. Such supervision is hard to obtain. The general impression is that an engineer is not needed until profile, plan and specifications for macadamizing are to be made. It will doubtless require mandatory legislation before intelligent technical men will be placed and retained in such positions.

As to the character of roads to be built. If near a large town, where the traffic will be considerable, the road must be of generous width, 20, 25 or even 30 ft. for the metaled portion. For country roads, where the traffic is less, this would be a waste of space and

material, and a width sufficient for two vehicles to pass is enough— say 12 ft. Similar considerations will govern the selection of material. Where the traffic is heavy it will be wise economy to secure the hardest, toughest stone available, even though a softer stone can be obtained near at hand at less expense. The cost of rolling a trap rock road will be greater than for a limestone road, but if the traffic is sufficiently heavy a higher price for the tougher material, as well as increased cost of rolling, will be justified by the more permanent results obtained. Mr. Lewis.

In cases where the traffic is light, good limestone will make a most satisfactory road, and one which can be more economically repaired than any other. Dependence will often have to be placed, however, upon the local stone, unless, as already remarked, the traffic is very heavy, when the toughest and best material should be obtained even at a considerably greater cost. Field stones are usually tough, and make an excellent road metal, but are apt to be of variable hardness.

As to the foundation: Drainage is a *sine qua non*. This is a very simple matter on Long Island, where the soil is nearly always gravelly, and surface water soaks away very rapidly. Side ditching and underdraining must be done, if necessary, to secure a dry roadbed. Once drained it can be rolled, and the character of the soil is, in the speaker's judgment, a matter of little consequence, except that if it is a loose, clean sand, the addition of a little earth or loam will be necessary before it can be rolled. After a thorough rolling with a steam roller of moderate weight, the roadbed is ready for the stone. The aversion of Macadam to a foundation of large stone was very strong. He said:

"The roads can never be rendered perfectly secure until the following principles be fully understood, admitted, and acted upon, viz., that it is the native soil which really supports the weight of traffic; that while it is preserved in a dry state it will carry any weight without sinking, and that it does in fact carry the road and the carriages also; that this native soil must be previously made quite dry, and a covering impenetrable to rain must then be placed over it in that dry state; that the thickness of a road should only be regulated by the quantity of material necessary to form such impervious covering, and never by any reference to its own power of carrying weight.

"The erroneous opinion so long acted upon and so tenaciously adhered to, that by placing a large quantity of stone under the roads, a remedy will be found for the sinking into wet clay or other soft soil, or, in other words, that a road may be made sufficiently strong artificially to carry heavy carriages, though the subsoil be in a wet state, and by such means to avert the inconvenience of the natural soil receiving water from rain or other causes, has produced most of the defects of the roads of Great Britain.

"At one time I had formed the opinion that this practice was only a useless expense, but experience has convinced me that it is likewise positively injurious."

Mr. Lewis. The following evidence given by him before a Parliamentary committee shows still more clearly what views he held on the subject :

"I should think that 10 ins. of well-consolidated material is equal to carry anything.

"Q. That is, provided the substratum is sound?

"A. No, I should not care whether the substratum was soft or hard; I should rather prefer a soft one to a hard one.

"Q. You don't mean you would prefer a bog?

"A. If it was not such a bog as would not allow a man to walk over I should prefer it. * * * I think when a road is placed on a hard substratum, such as rock, the road wears much sooner than when placed on a soft substratum.

"Q. But must not the draught of a carriage be much greater on a road which has a very soft foundation than over one which is of a rocky foundation?

"A. I think the difference would be very little indeed, because the yield of a good road on a soft foundation is not perceptible."

Of course these extreme views as to the unimportance of foundation would not be endorsed by engineers of to-day.

Some of the finest specimens of road making are those laid on a telford foundation—notably, the roads in Central Park and some of those in Prospect Park. It would not be prudent to say how thick they are, and they are perfectly drained, but their cost must have been very great. Assuming that a road is to be maintained after construction, why make it more than 8 ins., or, in many cases, 6 ins. thick? A road of this thickness made with two courses of broken stone will certainly sustain any load which it can be called upon to carry. The real value of a road, per mile, increases with its length, not with its depth, and the public money can best be expended by increasing the former.

In building country highways, city lots upon which to assess the cost are not available. With farm lands within two hours' ride of New York City, selling for from $2 to $20 per acre, in addition to the cost of the buildings upon them, it is not practicable to lay telford foundations and put on the broken stone by the foot. Macadamized roads 8 ins. thick and 12 ft. wide can doubtless be built in country districts, where material can be obtained within a moderate distance, for $5 000 per mile, and in many places for $3 500 to $4 000 per mile. General Roy Stone stated before the Senate and Assembly Committees at Albany that $1 000 per mile would build good roads in New York State, except in the vicinity of large towns, but he must have had in mind something other than what is commonly known as a macadamized road. The method of building, the size of stone, the rolling, etc., are so well known, and so much has been written upon the subject, that it is scarcely necessary to refer to them before this Society, except to promote discussion, for there are still some differences of opinion. Some engineers, faithful disciples of Macadam, still prefer stone as nearly as possible uniform in size, entirely free from chips

and stone dust, though they would scarcely specify, as did Macadam, that "The stone should be broken by men sitting on their haunches and striking it with a small hammer." This might be done within prison walls, by the State, but scarcely under other circumstances. Stones varying in size from 2 ins. to $\frac{3}{4}$ in. will have fewer voids, and will form a more solid roadway, with far less rolling than in the case of those of uniform size. When limestone is used, stones varying between the limits named will, with the addition of a few screenings of the same material, make a smooth, hard road, with comparatively little rolling, and one which will be very satisfactory under light or ordinary traffic, except in dry weather, when it will be extremely dusty. Efforts have been made to build a good road of trap rock, without the addition of any fine material, but the speaker believes they have been unsuccessful. Persistent rolling with very heavy rollers, preceded by a sprinkling cart, has reduced the size of the surface stone, but there has been no cementing of the surface material. The addition of trap-rock screenings and flour resulted in a great improvement. This flour is largely quarry sweepings (or was before the demand for trap became as great as it is at present), and the speaker is convinced that the more foreign matter the flour contains, the more satisfactory are the results. Specifications recently prepared provide for a binding material for trap-rock roads, 60% of which shall consist of trap-rock screenings and flour, and 40% of selected coarse sand or fine gravel, preferably mixed with a little clay or hard-pan. Mr. Lewis.

Limestone screenings as a filler with trap rock give excellent results, though if the road is to be subjected to very heavy traffic the trap screenings mixed with sand would probably be better. The use of the Roa Hook gravel from the Hudson River near Peekskill has given admirable results in binding roads, and in patching up the surface, and it makes a beautiful surface for riding, but it is short-lived and expensive.

If the State is going into the road-building business, there is a source of economy which may be taken advantage of. There are about 1 000 able-bodied men in Sing Sing Prison who might very well prepare material for the roads. In California it is said that the cost of furnishing road material by the State has been reduced to about 25 cents a ton. There, a vein of excellent trap rock runs through the prison grounds, and they have set the prisoners breaking it. The railroads have given favorable rates, and it is said they are laying stone along the roads at about the price at which it can be produced at ordinary quarries.

The important points in road building, as they appeal to the speaker, may be summarized as follows: The anticipation of the ultimate, substantial improvement of every road and the determination of the work which is to be done with that in view; the rectification

Mr. Lewis. of grades where possible, bearing in mind that the loads which can be hauled over any road are determined by the steepest gradient on the road; the proportioning of the width and depth of the road in accordance with the density and the volume of the traffic which it will have to sustain; a thorough and conscientious drainage of the foundation where the material requires artificial drainage; the selection of the toughest material which can possibly be secured for the money available; thorough rolling, with judiciously selected binding material, and, above all, the care and maintenance of the road when complete; and to these might be added constant supervision by a trained technical man, whose term of office shall not be one or two years, but who shall be kept there so long that the State or the county may have the benefit of his experience.

Mr. Tomkins. CALVIN TOMKINS, Assoc. Am. Soc. C. E.—About the year 1865 the Central Park roads were macadamized with chips from quarries near New York. These chips consisted of the waste material from the quarries, which had not been put through crushers. From that time down to about 1890, generally speaking, roads were built in any way by anybody. County commissioners usually built the roads, and very little attention was given as to how they should be constructed. Stone was bought, carted to the road, distributed, and allowed to wear down under the travel. Engineers, as far as they did have charge of road building, treated questions that came before them diversely, and there was no uniformity of practice. Since the early nineties conditions have changed very much. There exists now a general agreement as regards the practice of road-making, and the art has drifted more into the control of engineers and county commissioners, and politicians do less road building. There is now a skilled body of contractors willing to build roads, and as a consequence much better roads are built than were constructed formerly. There is one point, however, to which the speaker would call attention, where all that could be done has not been accomplished. Road building is now in the hands of engineers, but it has not as yet attained the position of a fine art. Almost any engineer accustomed to road work will build a good road; but the difference in the economies that can be effected in roads as constructed by some engineers and as constructed by other engineers is still very great. These nice economies should be discussed, rather than questions of general practice, which already are well established. The question of stone sizes, mentioned by Mr. McClintock, is a practical one. It is necessary to use all the product from the crushers if roads are to be constructed economically; otherwise the large stone, to get it down to a uniform size, must be recrushed, and the small stone will be wasted.

Another point in which nice questions of economy occur is in connection with the subject of thick or thin roads. As far as the speaker's

Mr. Tomkins.

experience goes the art of road building consists in making a thin road go just as far as possible. When bad places are encountered, over a morass or on a clay bottom, it may be necessary to put down a thick piece of road-bed such as will distribute a heavy load over a large surface before it reaches the limit of the bearing strength of the road. It is necessary to put down thick roads where they are subject to heavy, urban travel. It is not necessary, however, to build even 8-in. roads in country districts generally, particularly where there is a sandy bottom and the drainage is easily taken care of. If a little more is spent for drainage, where the bottoms are not so good, a thin road can be made to do as well as a thick one, and money will be saved. Such questions in road building are new and interesting. All the suggestions that have been made in this discussion are safe, so far as experience leads the speaker to judge of them. He thinks that it would be safe to go farther, in some instances. The principal consideration, which should be borne in mind, is: How far can these roads be safely constructed? Is it not also worth while sometimes to make an experiment, when it does not cost much, in order to see what it is possible to do in the direction of making the minimum amount of money build the maximum amount of road? As Mr. Lewis truly said, a road is valuable to the people who use it, not according to its thickness, but according to its length and width. In this connection the speaker does not believe—and this is the only technical point he wishes to make—that enough attention has been given to the question of consolidating roads into compact, almost monolithic, masses of stone. A road which will shed the water to the sides and keep its base dry conforms to Macadam's idea of what a road should be—simply a wearing surface on top of a dry solid underpinning of any character. If this base is kept dry by protecting it on the sides with ditches and drains, and by constituting the road-bed surface virtually a roof to shed the rain, the road is just as effective from an engineering point of view as a sound tooth, which simply consists of a hard surface upon a soft base. In order to accomplish this it is desirable to consider first the question of filling and thoroughly rolling the road-bed; and, secondly, of cementing the pieces of stone together and keeping them in that solidly set condition regardless of wet or dry or windy weather.

The question of suitable materials for these purposes is most important. Trap rock is undoubtedly the densest and hardest material which can be used, but it lacks binding properties, and, when used by itself in thin roads, particularly if not carefully rolled, is likely to ravel out and go to pieces.

Even when thin granite or trap-rock roads have been carefully consolidated during construction, they are likely to go to pieces in dry, windy weather, or when subjected to unusually heavy loads, or when the soil becomes wet and yielding beneath them.

Mr. Tomkins. This question of maintaining the surface of thick granite or trap-rock roads, and of maintaining the body of thin granite or trap-rock roads, is a serious problem which confronts engineers.

In the speaker's estimation, thin roads subjected to light travel can be built more cheaply and maintained more effectively when constructed of other material than trap rock or granite, since it is possible to cement such roads together, when it is not possible to do this with the harder stone by itself.

It is possible, however, to bind a granite or trap-rock road together with cementitious limestone screenings in such a way as to bring into play the maximum efficiency of both stones—the wearing qualities of the trap and granite, and the cementing qualities of the limestone. In such roads, when properly consolidated during construction, the pieces of harder stone do not become separated from the mass, and the surface itself remains smooth, the wear coming upon the surfaces of the harder stones, which are ground flat, and which do not project in a rough, knobby manner, as is likely to be the case when the siliceous binder is washed out of roads constructed of siliceous stone.

Many examples of such trap-rock and granite roads bound together with cementitious limestone can be seen in the immediate vicinity of New York City, some of which have been subjected to travel for a long time, and give evidence of unusual wearing qualities, besides affording a smooth surface.

This question of binding thick and thin roads, however, is only an instance of the general proposition which the speaker wishes to make, namely, that the present interest, which this road-making question has for engineers, commissioners, tax payers, and others, will be found in the consideration of economies to be practiced in construction and maintenance, rather than in the general principles of construction itself, which principles are now pretty well established.

An engineer who can build roads for $4 000 a mile as effectively as another engineer whose specifications make their cost $8 000 a mile, will come to be considered by the community twice as good a man as the latter.

Mr. Roullier. G. A. ROULLIER, M. Am. Soc. C. E.—The cost of road construction is a question so extremely important that it requires special consideration. Upon it depends the expansion of good road work throughout the country. Probably the principal item in which economy can be practiced—because it is about the only flexible item encountered in road construction, the majority of the items being fixed—is the thickness of the road-bed. Still, the speaker believes this is the very item on which engineers disagree the most.

The question is, how thin can roads be safely built? It has not yet been positively ascertained, as far as the speaker knows. Having made some experiments in that special direction he would like to sug-

gest this principle for consideration, so that a minimum thickness may be arrived at, *i. e.*: A road-bed having been constructed strong enough to safely carry the traffic that it is intended for, the condition of the surface becomes absolutely independent of the thickness. The condition of the surface will be dependent on suitable design and proportion, good workmanship and intelligent maintenance. By this is meant that, if the thickness of the road-bed has been so proportioned that it will carry the traffic without being cut through, and a certain factor of safety added to that thickness, further increase will certainly not insure a better surface. Hence, why should a certain limit of thickness be exceeded, providing that limit can be determined? Of course, in thin road construction—that is, construction without artificial foundation—a careful distinction must be made between roads intended for light suburban traffic and those designed for main thoroughfares. Mr. Roullier.

In the construction of thin roads the speaker must differ from Mr. Owen in the proposition as to the binding or filling material. The speaker has found that in building thin roads it is almost impossible to get successful work with a soft binding material. It is absolutely necessary to use a binding material that is hard and coarse, so as to prevent any possibility of motion in the mass. If motion occurs in a thin road-bed, failure is bound to follow. In the filling of these roads, the speaker would suggest that sand and coarse screenings will, as a rule, give the best result. He has tried almost every material, and believes that a great many of the failures in thin roads have been caused through the use of a soft filler. In filling these roads he has had considerable success by adding to the filling of sand and trap-rock screenings a slight amount of clay in some cases, in others limestone dust, adding about 30% in each instance. The raveling in those cases was materially reduced, and the road-beds could be left exposed to the weather without detrimental effect.

It is almost impossible to keep roads surfaced in localities where there is heavy traffic, but where the traffic is not too dense or too heavy a very good covering can be made by simply using the dust of the crusher with a very slight amount of clay; the speaker has seen it tried in some places quite successfully. He thinks that any one who will give personal attention to it for a short time will find that on such roads a covering can be placed which will remain there and give but little trouble.

E. W. HARRISON, M. Am. Soc. C. E.—On the Hudson County Boulevard, following the lead set by Mr. Owen in Essex, N. J., about 400 000 sq. yds. of telford macadam have been laid in the last five years on roads whose conditions were those of city streets. Heavy traffic had to be provided for, and also the sewerage. The drainage had to be arranged so as to connect with the sewers already con- Mr. Harrison.

Mr. Harrison. structed by different municipalities, and with a view to future construction. Under the Act, no sewers could be built except on the line of the Boulevard. The road is theoretically 12 ins. deep, but practically about 14 ins. There are 8 ins. of telford, of ordinary specification, all trap rock, and 4 ins. of macadam. It is endeavored to have about 2 ins. to 2½ ins. of stone which will pass through a 2½-in. ring. On top of that is put the 1½-in. course, and the surface is brought up with coarse screenings about ½ to 1 in. above the finished grade, using a great deal of flour and wetting the road all the time and keeping a roller on it.

When work on the road was begun the cry of the contractors was, "You cannot expect to bind trap rock with water." In the first 8 miles built there is not a spoonful of anything but trap rock, but a great deal of rolling was done and a great deal of water used. This section is standing much better than any other portion of the road.

On the upper part of the road the water had to be procured from a private corporation, and was very expensive. The road was built in summer. The water evaporated about as fast as it went on, and in this place the speaker allowed the use of a slight powdering of clay between the 2½-in. and 1½-in. layers. The clay was needed to hold the water a few seconds only while the roller was going over the road. There has been no trouble from raveling. Of course, this road is sprinkled. In a country road, lime or clay might be used, which might prevent the raveling. The greatest trouble is to keep the road free from the dirt of unpaved streets, from which there comes a great deal of wash.

A wagon coming through those unpaved streets loads its wheels with mud, and the moment the horse strikes the telford he begins to trot and the mud drops on the road. Some of the contractors claim that this has helped to consolidate the road, and there are people in Jersey City who claim that it is a good top dressing. The greatest difficulty experienced in building the road has been to accommodate the top surface to the use of bicycles, trotters, riding horses and heavy teams, and the speaker has come to the conclusion that it cannot be done so that everyone will be satisfied. The speaker believes in bringing the traffic right down on the stone, but if all the top dressing is stripped from the road so as to bring the heavy traffic on the stone, the wheelmen object.

In this Boulevard there is one peculiarity which has often been criticised. On the fourteen miles from Bergen Point to the county line, the middle of the road is gravel. That was a radical experiment. It was desired to get as wide a road as possible with the money available, and at least $1 000 000 had to be spent for right of way. Forty houses were bought and 400 were moved, and almost the whole right of way had to be condemned. Bridges over the railroad cuts cost $130 000, and it was found that the road

could not be built at the prices then ruling and give the full 60 ft. width of macadam. The problem was whether the example of Essex in putting 18 or 20 ft. of macadam in the center with earth wings should be followed, or whether the earth should be placed in the center and the macadam at the sides. Just before that time the late Chas. B. Brush had built a short piece of road in the upper part of the county, which was intended to have a trolley in the center. He had put macadam and telford on the sides, and left 15 ft. in the center for the trolley track. This road had been in existence about two years, and the center was perfectly drained by the telford, was dry in the spring and was used in preference to the stone road for fast driving. The speaker took that idea and enlarged upon it a little. A 15-ft. strip was laid out in the center of the road, and after preparing a subgrade 4 ins. below the finished grade, or at the elevation of the top of the telford stone, a 4-in. layer of Shark River gravel was placed on the southern 8 miles of the road. The upper 6 miles has 4 ins. of Roa Hook gravel. The Shark River gravel has now been in place 3 years, and has been worn down 1½ ins. In the spring the gravel center, with the wings of telford going down a foot below, dries up quicker than the macadam. Gravel dries very fast, and the speaker has a great deal of faith in the South Jersey gravel roads, provided they are well drained. It is a very great mistake to haul trap rock to Cape May, where there is good gravel. Mr. Harrison.

There is one other point in regard to which the speaker has not heard, except in the Massachusetts case. In constructing this road the surface was first shaped and rolled hard; then the telford was put down, and after breaking off the points and wedging it well, the steam roller was put right on top of the rough telford. Two Harrisburg rollers and two English rollers were used. Those in charge of the English rollers went right ahead, but those having the Harrisburg rollers were at first afraid it would smash the rollers. This rolling is a very great success, because the bottom telford is put down so that it never comes up again, and if there is any weak point it is at once discovered. The real test of the telford was the steam roller. When the contractor was willing to have his telford rolled and made a generally smooth surface, the speaker was satisfied that the other course should go on. Loose chips were not allowed on top of the telford, although the speaker does not know whether they would have done any harm or not; but he had an idea that in rolling they might work up through the broken stone. The contractors all wished to throw chips on and make it easier for the rollers. The telford was rolled; the first course was rolled, also the 1½-in. course; screenings were sprinkled on it, and it was finally rolled with water until the surface puddled.

The speaker has found by the mileage of the rollers that every inch

Mr. Harrison. of the road has been gone over from 100 to 115 times, and he thinks it has paid and has been the making of the road. Since the road was finished it has been in the hands of the freeholders for three or four years. The great problem is its management and care. The speaker's idea is that a great many men can be found who can build a good road, but the hardest problem is to find one who can take care of it after it is built. Most people think that when a road is built it will last forever; but there must be a certain percentage of material put on it every year, and put on in the proper way. In regard to one part of the Boulevard, after it was finished a complaint was made by the inspector that for about a mile the macadam was too thin. This portion was very finely surfaced, bound in nicely, and was below grade and a little thin, and, without picking the surface, 1½-in. stone with screenings was put on to bring it up. It was rolled thoroughly, and that portion of the road to-day is about as fine as any other in the whole 20 miles.

Mr. McLaughlin. J. J. McLAUGHLIN, M. Am. Soc. C. E.—There are in Queens County about 800 miles of macadam roads and streets which have cost about $10 000 000. Of these, 335 miles are macadam roads built to carry the traffic of Long Island farmers. There are more than 3 000 wagons running to market every day for very nearly six months in the year. Each weighs when loaded between 10 000 and 11 000 lbs. The wagon itself as it stands weighs 2 600 lbs.; 3-in. tires are used. For this traffic 4-in. roads are not built. The heaviest roads are 8 ins. thick in the center and 6 ins. at the sides. There are now 175 miles of these roads under construction. As far as the sustaining power of a thin road is concerned, the speaker would like to corroborate what Mr. Roullier has said, and can give a good example of the application of his theory. In the village of Jamaica, in 1897, there was a stretch of street about ¾ mile long that paralleled two roads then being improved. The work was done in the spring of 1897 after a sewer had been laid in the same street. It was built with 4 ins. of stone spread loosely. The stone was measured by blocks, and was spread 4 ins. thick, loosely, and then rolled. From actual count there were from 80 to 100 of the heavy loads just mentioned per day for over three months running over this piece of road, and in only one case, where there was some careless work over a sewer trench, was there any material sign of wear, and in no place a break. There was not a square yard of the street broken, and it carried these wagons.

In consideration of the fact that so much has already been said about sizes of stone and methods of construction, very little which would be of any material interest can be added. The speaker, however, would like to back up Mr. Harrison's assertion, even though the discussion of maintenance is to be debarred, that a road may be built poorly, but if there is an adequate system of maintenance put right behind the construction, the road can be saved and made very good.

In Queens County there are some roads known quite widely as being among the best bicycle roads, but which were poor when built. They were made good roads by efficient maintenance. Queens County has the best system of maintenance in vogue in the vicinity of New York City, and it is considered of the greatest importance. After a road is well constructed it is immediately passed into the hands of the parties who maintain it. Mr. McLaughlin.

The method used is practically as follows: The roads are divided into sections; the length of a section depends on the location and the character of the work, and averages about 12 miles. Each section is in charge of a foreman who keeps a small gang of men, generally two and in some instances as many as four, continuously employed. These men keep the stone work slightly covered with a mixture of gravel and loam, mend slight depressions, keep the gutters cleared up, and the wings or earth shoulders free from grass and weeds. These earth wings are ploughed and reformed when necessary, in order to provide earth roads and good drainage. The wings are generally wide enough to allow of the passage of wagons on either side of the stone work.

In addition to the foreman, there is a superintendent who lays out the work in detail and generally oversees the operations. This was formerly the system throughout the county and is still used in the eastern portions. The work in the borough towns has been let by contract. Contract maintenance does not appear to be a success.

The county owns its own steam rollers, sprinkling carts and machinery. The system has proved very efficient and at the same time economical.

George W. Tillson, M. Am. Soc. C. E.—In the construction of a macadam road there seems to be more opportunity for the engineer to show his good sense and ingenuity than in that of almost any other kind of pavement. If a street is paved with either asphalt or stone, certain conditions must be met. The asphalt must be of about the same thickness, and the stone blocks of about the same size. By varying the foundation according to the traffic, the expense may be increased or decreased, but in the main the engineer is bound to general conditions. With macadam, when the amount of traffic which the road must stand is known, it should be possible to construct a pavement that will meet the requirements very nearly. The three things which the engineer must study, and which will govern the expense materially, are the character of the stone to be used, and the thickness and width of the roadway. The stone selected need not necessarily be absolutely the best, but should be the best to meet the special conditions. Of course, it is understood that trap rock will stand more wear than any other material used in the vicinity of New York City, but in a great many cases it is better, in the long run, to use local stone which is not quite as good. That is, it will be cheaper to use a stone Mr. Tillson.

Mr. Tillson. that can be obtained at a small expense, and repair the road more frequently, than to take a material whose first cost is greater. On account of its hardness, trap rock is very difficult to bind; and, conversely, a softer material binds more easily and more quickly. Another disadvantage of trap rock is that when broken it is so far from being cubical. Most of it is flat, and, consequently, the size used for the upper course is often very difficult to bind. For streets or roads where the traffic is light it is better to use a softer stone, such as limestone, that will bind quickly and give a smooth road, than to use trap which requires heavy traffic to keep it in good shape, unless the road is remarkably well made. The binding material used with the trap is apt to wear out and be blown away, leaving the surface of the road in such a condition that in passing over it the wheels of a vehicle jump from stone to stone instead of bearing continuously on the surface.

The speaker has in mind a road that was built seven or eight years ago, of which a portion, at least, is of limestone, which has had almost no attention in the way of repairs. About three years ago this road was extended, trap rock being used. For the first year or two, when the surface was covered with Roa Hook gravel, it was almost an ideal road for either driving or wheeling. During the summer of 1898, however, no attention was paid to the surface, and now in passing over it the wheels jump from stone to stone. For that reason when the travel is to be light, and the road pleasant for riding or driving, especially in these times of bicycling, it is better to build the road of limestone than trap, although the limestone will wear out sooner; on exactly the same principle that one buys a calfskin shoe rather than a cowhide boot.

After the engineer has decided on his stone, he must determine on the width of the roadway. If intended only for travel continuously in one direction, it should not be wider than is economical for that purpose. If two lines of travel uninterrupted are to be provided for, the economical width for this purpose must be determined. Two trucks with 5-ft. width of wheel base, and 9-ft. width of load can pass on a 16-ft. roadway with a clearance of 1 ft., assuming the outer wheels of each to go within 6 ins. of the edge of the macadam; and it is seldom that such a necessity occurs on a country road. If one truck is not loaded it can turn out without difficulty. It seems, therefore, a waste of material to build a country road more than 16 ft., and in some instances more than 12 ft., wide.

A broken stone road, as well as a pavement, is made up of two parts, the foundation and the wearing surface. In determining the thickness of the foundation, the engineer should be governed by the weight of traffic that is to come upon the road, and make it strong enough to sustain that traffic and no more. It is as much the business of the engineer to avoid making anything too strong as it is to avoid

making it too weak. He makes almost as great an error if he constructs a road, or any other structure, costing 50% more than it should, as if he made it so weak as to necessitate an extra expenditure of 50% to make it good. The foundation should be made of a material which will not decay under the action of the weather, and which will be good and solid under ordinary treatment. If this is accomplished, it does not make any difference of what the bottom course is made. Mr. Tillson.

In determining the thickness of the wearing surface, the engineer must find out how much that surface will wear, and how much the surface can be permitted to wear without allowing the road to get too rough, and then make the thickness a little more than that permissible wear. By that is meant that if a road, after it has worn, say 3 ins., has worn so unequally that it must be entirely resurfaced, it would be a waste of material to make that wearing surface much over 3 ins. in thickness—say, make it 4 ins. If it is made 5 or 6 ins. thick, there is a thickness of 2 or 3 ins. below the available wear which is of no benefit at all and is waste.

After the wearing surface has been put on, comes its binding, which, in the speaker's opinion, is the most important part of road construction. The great trouble with many, or perhaps most, macadam roads is that they do not receive sufficient rolling. It is very common for a contractor to want to consolidate the upper part, perhaps the upper inch, of his wearing surface without any regard to the lower, so that only the surface presented by the road at the time it is finished is good. A great deal of care is necessary to prevent his doing this, or, if the road is being built by day labor, for the engineer to accomplish the other result, *i. e.*, to fill all the voids between the stones so compactly as to make the wearing surface one homogeneous and waterproof mass. The stone ordinarily used for this service, approximating what is generally called 1½-in. stone, has about 50% voids, and under the roller perhaps will compress one-half of that, leaving about 25% to be filled with the binder. The speaker has had some experience with different kinds of binders for trap-rock roads, and does not see why it is not proper to use any material that will bind trap rock securely and keep it bound. The specifications generally call for the road to be bound with the same material that forms the surface. On one road of trap rock the speaker used trap screenings, and it took a very large amount of rolling to consolidate it. On one end of the road, for about a quarter of a mile, a mixture was used, consisting of perhaps 25 or 30% sand, and the balance trap-rock screenings. That portion of the road was consolidated in about a quarter of the time that would have been required had trap screenings alone been used. A firm, smooth surface was obtained, and, although it has not been down more than six months, it has not shown any such signs of ravelling or going to pieces as the part covered with screenings only.

Mr. Tillson. In working trap rock the speaker found it necessary to use practically the method described by Mr. Cudworth, *i. e.*, to keep rolling and sprinkling the screenings at the same time. After the interstices of the stone are filled completely, if a substance that is practically all flour be added before the sprinkling and rolling are discontinued, a very much better result will be obtained, because unless a sort of a paste or mortar can be brought to the surface, it is almost impossible to get the trap rock solid.

The amount of rolling required for different roads is quite variable, and is governed by a great many conditions. In the reports of the City of Glasgow for the years 1896 and 1897, it is stated that something over 300 000 sq. yds. of road were kept in repair for those two years. The amount of stone used, as nearly as the speaker could estimate it, averaged about 3 ins. over the entire surface, and the amount of rolling on that varied from 200 to 2 200 sq. yds. per day. The amount of rolling per day on each street is given. Of course it is pretty hard to tell just why the amount of rolling should be varied on practically the same kind of work. The total cost of the repairs averaged about 12 cents per square yard over the entire surface.

There is another point in rolling, about which the speaker has never been able to get any satisfactory information, and that is the different effects of rolling at different speeds of the rollers. The French in some cases speak of rolling so as to get a certain ton-mile per square yard, ranging from .4 to .6 ton-mile per square yard. Taking this for a standard, if the speed of a roller is increased from 4 to 6 miles an hour, its efficiency is increased directly 50 per cent. The speaker, however, does not think that a roller will do anything like 50% more work if the speed is increased 50 per cent. To consider it the other way, as a matter of time, will a roller moving at the rate of 2 miles an hour, do as much work as it will at the rate of 4 miles an hour? The speaker does not believe it will. If not, what is the difference? By increasing the speed the efficiency of the roller is increased, but by what amount the speaker has not been able to learn. Experiments to determine the relative efficiencies of different rollers at different speeds would be of great value. The speaker had an account carefully kept of the amount of rolling done on one of the roads before mentioned. In the top there were about 18 000 sq. yds. The lower course consisted of 4 ins. of limestone and the upper course of 4 ins. of trap rock bound with trap screenings; and the area rolled per day averaged almost exactly 200 sq. yds., including both courses.

In the summer of 1898 the speaker rode over a road in Morris County, New Jersey, which seemed particularly good. He afterward learned from the engineer in charge, that the road was 4 ins. thick, that it was constructed entirely of trap rock, binding material and all, and had been down for four years. The speaker saw

it at the end of a dry spell of four weeks, and there was no sign of raveling or other faults. The engineer said the road was rolled in two courses, the lower course 2½ ins. and the upper 1½ ins. thick, and that each course was rolled absolutely solid. He said that a road of that character, 12 ft. wide, could be built at the rate of 100 to 150 ft. per day. That would be from 130 to 200 sq. yds. per day, and the speaker does not believe that a trap-rock road can be put down very much faster than that, and at the same time receive the necessary rolling. If Mr. Harrison consolidated the Hudson County Boulevard by rolling it 110 times he did well. Mr. Tillson.

CHARLES H. GRAHAM, M. Am. Soc. C. E.—Referring to Mr. Tillson's statement that limestone roads are more pleasant for driving and riding purposes than those constructed with trap, the speaker desires to express his agreement therewith, having in mind some old roads which were resurfaced in 1895. These roads were built originally with trap-rock, and were thin macadam roads without telford foundation. Being badly worn, they were remacadamized with Tomkins Cove limestone. The old macadam was spiked up by the roller, and the limestone was spread and rolled down. These roads have given excellent service, and are in very good condition to-day, except where they have been partially destroyed by sewer construction and openings for various kinds of pipes. Mr. Graham.

The speaker's experience with limestone has been that without some binding material it is not possible to get it to bond within any reasonable length of time, without an excessive amount of rolling, so much rolling in fact as to sometimes crush the stone. In handling limestone for macadam, the speaker has made it a practice to use loam as a binder, being careful to use a clay loam, and not one which contains any considerable percentage of sand. Water is freely used, and the macadam is thoroughly rolled until the excess of binder runs on the surface ahead of the roller in the form of a thin grout, which by continued watering and rolling is finally forced from the surface of the macadam. A thin coat of screenings is then applied and rolled, after which a wearing coat of screenings is spread. This method gives very good results.

Though the speaker has made no experiments to ascertain whether it is true or not, he believes that the most efficient work is performed by steam road rollers when they are run at maximum rates of speed.

HOWARD J. COLE, Assoc. M. Am. Soc. C. E.—In Essex County, N. J., which contains the cities of Newark and Orange and the residential townships of East Orange, South Orange, Montclair, Bloomfield and Nutley, a park system has been inaugurated, and among the first constructions have been the roads. Mr. Cole.

The type of road adopted as most serviceable under existing con-

Mr. Cole. ditions is a telford-macadam, 8 ins. thick, consisting of a foundation 5 ins. deep, and 3 ins. of 1½-in. broken stone and screenings.

The subgrade was carefully rolled, after which the telford foundation was placed by hand and firmly chinked and then set in place by a 10-ton roller. After this, a coat of binder, barely ¼ in. thick, was placed on the telford, and then the 1½-in. stone was spread. The roller was again used until the small stone had been rolled to a good surface, when another thin coat of binder and screenings was spread, after which the road was watered and rolled until a satisfactory surface was secured.

The work so far has been constructed entirely from the trap rock of the Orange Mountains, and after nine months' use is in first-class condition. The cost of this work was 59 cents per square yard. About one mile of the roads of the park system has been built, but the plans call for the construction of about 40 miles of drives, of substantially the same character, in the parks and connecting parkways.

In the Mountain Reservation some earth roads are being built for temporary use, until the permanent administration roads are built. These roads, which are about 12 ft. wide, follow generally the lines of the old wood roads, with slight changes of alignment and grade, the cutting of any trees or shrubs being avoided as much as possible. The road is given a moderate crown, and good drainage gutters are made on each side, while at frequent intervals the water is carried off by cross-drains and culverts, the main object being to keep the road as dry as possible. During the past season these temporary roads have been used considerably by pedestrians, bicyclists and the driving public and, with but little attention to their maintenance, have stood the traffic well.

In the South Mountain Reservation, near Milburn, N. J., the Park Commission has assumed the care, custody and control of an old public road (Brookside Avenue) for a length of two miles. This road has been regraded and then dressed for a width of 12 ft. in the center with a coating of 1½-in. stone, which, after it had been compressed by a 2-ton horse roller, was about 2 ins. thick. After being thoroughly rolled, screenings were spread and the road again rolled. The cost of this improvement was $1 000 per mile for grading, and $1 000 per mile for broken stone. Thus, at a very reasonable expenditure, a dirt road was converted into a desirable road for pleasure driving.

Mr. Seaman. H. B. SEAMAN, M. Am. Soc. C. E.—Mr. Tomkins has proposed trap-rock roads with limestone binding, and Mr. Tillson has described a trap-rock road which was worn so rough that the wheels of a vehicle would jump from point to point of the larger stones. These two conditions illustrate cause and effect. The softer binding material wears down first, and leaves the harder stones projecting.

In replacing a number of macadam roads, the speaker found occasion

to use trap rock, with screenings of the same material. Water was used freely in rolling, and no trouble was experienced in getting the material to bind properly. The road was smooth as long as it lasted. Mr. Seaman.

GEORGE HILL, Assoc. M. Am. Soc. C. E.—When engaged on the geological survey of New Jersey, the speaker saw probably 20 miles of road apparently made of blue shale which had been broken into small sizes and dumped on the foundation prepared by breaking off the outcropping rock on the side of the road, the traffic working it down. Those roads were only slightly muddy, when the roads in the valleys and along stretches of ground untreated were practically impassable on account of the mud. They kept a good surface throughout the year, apparently without any special attention being given to drainage, which was probably secured through the undulating character of the road, and the numerous streams which crossed it. Mr. Hill.

On the seashore many miles of gravel surfacing are used, the practice being to shape up with sand, and surface with 4 ins. of clayey gravel; the gravel ranging in size from $\frac{1}{8}$ in. up to $1\frac{1}{2}$ ins., and the percentage of clay varying with the amount naturally present in the bank from which the clay is taken. Both surfaces are shaped with a road scraper of the simple construction usually found in country places, the more elaborate road machines being but seldom used. For compacting the surface, reliance is placed on the light traffic, and the passage of the carts which bring the material, and which are usually provided with 4-in. tires. To be kept in order the roads require constant attention, and resurfacing about every second year. No special provision is made for drainage, since the sand is pervious and consequently dry.

S. C. THOMPSON, M. Am. Soc. C. E.—Mr. Hill has covered a question which the speaker was about to ask, *i. e.*, what would be done in a country where there is no limestone or trap rock. The discussion has brought out pretty thoroughly what can be done in the way of a macadam road with limestone, or with trap. The speaker has lived in a section, however, where there is neither one nor the other, but where it was necessary to make as good roads as possible. The custom there was to make them of two kinds of gravel; one practically a dry gravel free from any admixture, and the other a hardpan. It was found that by mixing the two, making them thoroughly wet and using a 4-ton or 6-ton grooved roller, with a 4-in. face and a 4-in. groove, for combining them, and by going over the work repeatedly and finishing it with a smooth roller, that a surface could be obtained which would not only stand heavy traffic, but would also wear well and shed water. It made a very compact roof. In one case where the speaker had occasion to build an embankment for a special purpose, the gravel was mixed in that way; the roller and a water cart were kept going all the time, and the embankment was left over winter to provide for settlement. It was tested carefully, and in the spring Mr. Thompson.

Mr. Thompson. it was scarcely possible to discover that there had been any settlement; showing that such material could be used for roads and give excellent results. Of course, such roads do not wear as long as the macadam roads made from trap or limestone.

In the eastern part of Massachusetts there are a great many miles of those gravel roads, in many cases built with no hardpan at all, which stand the traffic admirably and do very good service, and the expense of building them is trifling. The ordinary way of building is simply to excavate, get a roadbed of sufficient width, fill in with gravel and roll it. In many cases the traffic itself takes the place of the roller. In that way a hard bed, and one that gives very good results, is obtained.

Mr. North. EDWARD P. NORTH, Vice-President, Am. Soc. C. E.—There seems to be some difference of opinion in regard to the relations between the foundation of a macadam road, and the treatment of the wearing surface. One engineer has said that he did not care for binding, and others have laid a great deal of stress upon it. The speaker would be justified, perhaps, in calling attention to the fact that the engineer who did not care for binding proposed a telford foundation, and the engineer who laid great stress on binding proposed a thin road on an earth surface. In building a thin road it is necessary to use the best possible binding, so as to produce a road that is both a wearing surface and a roof. With a telford foundation, however, this is not so absolutely necessary, as the stone, without being so well bound, will still make a tolerably good road.

A point which has been overlooked in this discussion is the influence of roughness in the surface of the stone used, on the ease with which it can be compacted, and on its general wearing qualities, not only in its freedom from "raveling," but in its ability to stand wear when kept so well watered as to resist breaking up from dry winds. Some stone with a decided "tooth" that is not otherwise desirable makes a very good macadam. In Saratoga Springs all the macadam streets are in very good condition, although the stone used is a calciferous sand rock (a sand stone bound by lime instead of silex), which is so weak that the edges of spawls can be broken off by a pressure of the thumb. On Broadway, a street that has a considerable amount of heavy traffic, as well as much fast driving, there was laid a 6-in. macadam on the old dirt roadbed, which lasted, with some maintenance, between 6 and 7 years before it required resurfacing.

The tooth of the stone undoubtedly had great influence in its resistance to wear, by checking or preventing the tendency of the stones to rub one another under passing loads, and the presence of a lime cement was also valuable, for it is doubtful if a pure sand stone of that strength would have worn as well. On the eastern face of the Palisades a stone decidedly harder than the average trap is found. It is called

"black jack," irrespective of its color. That stone, which is very smooth, is condemned by road-builders on account of the difficulty of compacting it and making it stay in a road, even after it is well puddled. Mr. North.

There will be substantial agreement in the statement that, if it is possible to make a thin road which will not break up, the stone will wear longer than if on a rigid foundation, because a stone cannot be easily broken by striking it when on soft earth. It breaks very easily, on the other hand, if struck when on a hard surface. So, between Telford and Macadam, Macadam was right in proposing to build a thin road. Material will wear longer without a telford foundation, if it is not forced into the subsoil by the traffic. This leads immediately to the consideration of the drainage of the road.

As long as the road stays dry and compact it is almost impossible to crush 2 ins. of well-compacted macadam into the soil. The speaker has seen a load weighing 2 or 3 tons on narrow-tired wheels drawn over such roads without injuring them. If it is possible to drain the roadbed, or to so shield it with the macadam that it will stay dry, it is a waste of material and a waste of money to build a thick road. If the roadbed cannot be kept dry enough to prevent the traffic from crushing through, of course more money must be put in the foundations. All engineers should remember, however, that monuments pay no interest, and however desirable they are as a matter of piety or beauty, monuments that cannot be seen are worthless, and that, as Mr. Lewis says, it is the length of the road that is valuable, and not the depth. Hence an engineer should make his brain save material by doing better work.

Mr. McClintock has said that in Massachusetts they put a certain thickness of porous gravel between their telford and the roadbed. The speaker is not satisfied with this practice. On a clay road, or on any road that is muddy in wet weather, the finer the material that is placed on it, the smaller will the apertures in the material be, and the more certainly will it prevent the mud from coming through. The speaker has used, both for highways and for railroads, a layer of very fine sand; in one instance sand so very fine that it was blowing sand. Five or six inches of it were put in clay cuts which were undrained at the time and stayed undrained for three years after, and broken stone or gravel was placed on the sand. The roadmaster said, at the end of three years' experience, that he had less trouble with the track on the sand in undrained cuts than he had with the track on embankments where there was no sand. A colander, if put in a bucket of water will sink, but if it is wrapped in cloth, or anything impervious to the water, it will float. So a macadam roadbed will float on an undrained clay surface, if material that will not permit the clay or mud to go through is placed between the two; whereas if coarse stones, macadam or tel-

Mr. North. ford, are put on soft material, as was the practice in Hartford, Conn., for instance, before Mr. Bunce took charge of the streets, the clay can be rolled up through the telford or macadam. The speaker believes this was done almost invariably in Hartford, and then the road was left as finished.

Mr. Pierce, of Bridgeport, built some 70 miles of 4-in. trap road, which was puddled or bound more with sand than with screenings. Of that 70 miles about half a mile broke up in the course of seven years. It broke through in two places, and there he added 3 ins. of stone. These roads lasted, with more than ordinarily heavy traffic for a town of that size, as he said, for seven years before it was necessary to resurface any of them, though, of course, they required some maintenance. His practice is interesting and instructive, as he did not underdrain his roads, nor did he use sand or other material impermeable to mud under the broken stone, but he puddled his roads carefully and kept his ditches open all the time. In the spring, when the grass started, he waited for a hot day and spread sand to a depth of about ½ in. through the gutters, left it there for a day or so, then scraped it up with a road machine and applied it in other places. The sand, under a hot sun, burnt and killed the young grass and weeds in a short time. That was understood to be all the drainage that the 70 miles of road had, except right in the town, where there were sewers.

There are other circumstances besides the first cost of a road which should govern an engineer in selecting the style of road or the material that he is to use. In the City of New York, for instance, a street cannot be paved, with the approval of the law, unless an assessment is made for it. When a street is once assessed, it cannot be assessed again. This is true in both the Boroughs of Manhattan and the Bronx. It is not so in Brooklyn. The practice, therefore, in New York since 1868 has been to build nothing except telford roads. Some of them have been 16 ins., and some 22 ins.; none less than 16 ins. deep. These roads have this advantage: when the traffic gets too heavy for a macadam road the telford foundation is very good, either for an ordinary granite or trap pavement; or, with a little dressing up and filling in at places, for an asphalt pavement. So, possibly, there is engineering justification, as well as legal justification, for building these heavy roads in New York. Building such a pavement in the country, however, results in the unproductive investment of capital.

In Vermont the assessed value of the State amounts to only $1 200 per mile of road. The equalized value of real and personal property subject to taxation, in the State of New York, for 1896, was $4 368 712 903. If the estimate of 123 000 miles of road in the State is correct, improving all the roads in the State at the cost of $10 000 per mile, as has been advocated, would confiscate over 27% of the above valuation. If the values assessed against the counties of New York, Kings

and Queens are taken from the total valuation, as above given, only about $4 000 per mile would be left, on an average, after a $10 000 improvement had been completed. And it is in the poorer portions of the country that the high cost of transportation falls with the most crushing force. Mr. North.

It is submitted that an engineer, or any one else, who proposes to improve the roads of the State of New York at the cost of $10 000 a mile, makes a mistake. It should be borne in mind that in the early practice of railroading, the cheapest possible roads were built. Those were better than any highway, and, as they reduced the cost of freightage, they not only earned money for the company owning the road, but also added to the wealth of the surrounding country; so that railroads which were originally built for as low a price as $10 000 per mile have so developed the wealth of the country through which they ran that the increased traffic has justified several rebuildings of the road, while the freight charges have been reduced to 8 or 10% of what they were when the road was opened. This would have been impossible unless the railroad had presented a cheaper means of transportation than the highway, and it would have been equally impossible to build railroads like the Pennsylvania Railroad, for instance, out of the resources of the country when entirely dependent on wagons for transportation.

Engineers having charge of either a district or a single line of road are urged to consider the possible advantage of diffused rather than of localized expenditure; that is, to make an effort to immediately bring the roads of the entire district up to the best standard of usefulness practicable, increasing the expenditure yearly until the roads reach the desired excellence. While there are areas of clay or sand that may require an immediate large expenditure for broken stone or gravel on their roads, most of the roads on the Atlantic Seaboard can be put in good order for, say, ten months of the year with a very small expenditure per mile, most of which will be concentrated on one or more bad places. Future expenditures should bring the roads up to continuous usefulness. This is not intended as a plea for poor roads, but to make any expenditure serviceable to the greatest area possible, and to admit of the marketing of produce through a longer period and at a lessened expense. It will be admitted that the poverty of the farming communities is due to the high cost of transporting their produce to the consumers, and any reduction of that cost must add an equal sum to their gains, and, consequently, to their ability to pay for improved highways.

T. McC. Leutzé, M. Am. Soc. C. E. (by letter).—The State of New York appropriated $50 000 to be spent in experimental "object lessons" for the farmers and dwellers in the country. Only the Schenectady and Troy road is actually under construction, but surveys Mr. Leutzé.

Mr. Leutzé

and plans are ready for other roads. One road in the first stage of construction is hardly an "object lesson." Nevertheless, the number of applications that have been filed for the improvement of roads is beyond expectation, and shows that the Supervisors of the various counties are fully alive to the necessity of good roads, and are anxious to take advantage of the offer of the State to pay half the cost.

Thinking it may be of interest, the writer gives herewith a list of the petitions received, from which it will be seen how totally inadequate is the sum of $50 000 to carry out the desires of the various counties.

Counties.	Petitions.	Miles.
Erie	6	48
Orange	8	141
Rockland	11	38
Monroe	15	102⅛
Columbia	1	2
Schenectady	1	2
Oneida	1	2½
Onondaga	3	2½
Rensselaer	4	17⅞
Totals	50	356

This would involve— at $10 000 a mile roughly—the large appropriation of $1 680 000 on the part of the State to cover the petitions already filed, and as the Legislature cannot appropriate more than $1 000 000 at a session, it is evident that some of the petitions will have to wait.

It would be pleasant to be able to build a road as it ought to be built; that is, a standard road with perfect tile side drainage, proper grades and slopes, cobble gutters, fences on embankments, and a substantial thickness of well-rolled macadam—the writer's idea of thickness being 8 ins. on a thoroughly rolled subgrade. Such are the roads that nearly all desire, but as soon as the price is mentioned, the elimination of items and quantities is demanded. Drainage is grudgingly admitted to be necessary, the fences are omitted, the macadam shrinks to a thin veneer, and the result is a cheap road. Take the Schenectady and Troy road, for instance. In the first plans the writer provided 4-in. tile side drains, cobble gutters and 8 ins. of macadam 18 ft. wide. The grades were better than in the present plan, but the tiles and cobble gutters had to be omitted, the grades changed, as excavation and embankment ran too high, and the macadam reduced to 6 ins. in thickness and 16 ft. in width. The writer does not lay so much stress on the width as on the thickness, in the sandy soil prevalent in that part of the State. It was even demanded that the use of the stone peculiar to that vicinity should be used in the macadam—

"Aqueduct Stone," a shale that would turn to clay in the first storm. This was happily averted, and the present State Engineer will insist on trap rock covering, at least, on all roads where macadam is to be used. Mr. Leutzé.

The writer consoles himself with the theory elucidated by Mr. North in an article in the *New York Mail and Express*, in which he suggests the propriety of making a road during the first year, that a farmer can use six months of the year where he could only use it three months before; to improve it in the following year, so that the farmer would be able to use it nine months instead of six months, etc., etc., and in this way, if first-class roads could not be built, "make the punishment fit the crime."

The Columbia County road is an entirely new one. It leaves the old road through the Shaker settlement at Mount Lebanon to the left, and, by following the contour of the hills, connects with the Massachusetts road (improved) at the State Line, and secures a 5% gradient over the entire mountain into Pittsfield. As some of the gradients on the old Shaker Road are 20%, 19%, 17%, etc., the advantage attained can be imagined. Economy here again demands a gravel covering, splendid material being handy for haul.

This road, including wire fencing, thorough drainage, one 40-ft. span bridge, one 48-in. wrought-iron pipe culvert, numerous cross-tile drains, and about 16 600 cu. yds. of material to be handled in grading the road, will cost about $9 000 for 6 500 ft.

There was considerable trouble in persuading the Shakers, who own all the lands in the vicinity, to grant the right of way, but after months of persuasion they granted it. It seems strange that a people which has preached isolation for nearly one hundred years should object the very first time that this realization of their doctrine was possible.

Warren B. Travell, Jun. Am. Soc. C. E. (by letter).—During the past year and a half the writer has done some experimental work with thin coverings for dirt roads, which may be of interest to engineers having to contend with similar conditions. The work was done in the Borough of Manhattan, on Fort Washington Avenue, which begins at One Hundred and Fifty-ninth Street and the Boulevard, and runs northerly to Inwood. Until the summer of 1897 this was a dirt road, but lightly traveled in the best of weather and quite impassable in times of rain. The soil is a micaceous sandy clay. Mr. Travell.

During that summer the writer spread, on a mile and a half of the road, a thin covering 15 ft. wide, using a variety of road material, with the express object of determining the smallest possible cost at which a dirt road having light traffic could be rendered respectable in New York City. There was no grading done, the only preparation being a simple scraping and rolling, which it was customary to give

Mr. Travell. the road about every week. Beginning at One Hundred and Fifty-ninth Street, the sections were spread with material as follows:

Section.	Location.	Length.	Material.
1.	159th–163d Sts..	1 523 ft.	2-in. Roa Hook gravel.
2.	163d –165th Sts..	420 ft.	½-in. Roa Hook gravel screenings.
3.	165th–167th Sts..	554 ft.	¾-in. feldspar (no dust).
4.	167th–179th Sts..	3 157 ft.	2-in. Roa Hook gravel.
5.	179th–182d Sts..	813 ft.	feldspar screenings.
6.	182d –188th Sts..	1 652 ft.	¾-in. limestone (no dust).

The following are the itemized costs of the several sections:

SECTIONS NOS. 1 and 4.—2-in. Roa Hook Gravel.

Length	4 680 ft.
Width	15 ft.
Thickness	2 ins.
Cost:	
451 cu. yds. of material at $1.97 (delivered)	$888.47
Spreading, 213 hours at 25 cents	53.25
Sprinkling and rolling, 298 hours, at 50 cents	149.00
Superintendence, tools, etc	41.00
Total cost	$1 131.72
Cost per running foot	.2418
Cost per mile	1 277.02

SECTION NO. 2.—Gravel Screenings.

Length	420 ft.
Width	15 ft.
Thickness	3 ins.
Cost:	
60 cu. yds. of material at $1.97	$118.20
Spreading, 24 hours	6.00
Sprinkling and rolling, 15 hours	7.50
Superintendence, tools, etc	3.00
Total cost	$134.70
Cost per running foot	.3207
Cost per mile	1 693.29

SECTION NO. 3.—¾-in. Feldspar Stone.

Length	554 ft.
Width	15 ft.
Thickness	2 ins.
Cost:	
51 cu. yds. of material at $1.40	$71.40
Spreading, 22 hours	5.50
Hauling and spreading 12 cu. yds. of rotten stone earth	18.00
Sprinkling and rolling	17.00
Superintendence, tools, etc	8.00
Total cost	$119.90
Cost per running foot	.2178
Cost per mile	1 149.98

Mr. Travell.

SECTION No. 5.—Feldspar Screenings.

Length	813 ft.
Width	15 ft.
Thickness	2 ins.
Cost:	
70 cu. yds. of material at $1.40	$98 00
Spreading, 25 hours	6.25
Sprinkling and rolling, 32 hours	16.00
Superintendence, tools, etc.	4.00
Total cost	$124.25
Cost per running foot	.1528
Cost per mile	806.78

SECTION No. 6.—¾-in. Limestone.

Length	1 652 ft.
Width	15 ft.
Thickness	2 ins.
Cost:	
146 cu. yds. of material at $1.40	$204.40
Spreading, 48 hours	12.00
Hauling and spreading 27 cu. yds. of rotten stone, earth	40.50
Sprinkling and rolling	40.00
Superintendence, tools, etc	18.00
Total cost	$314.90
Cost per running foot	.1906
Cost per mile	1 006.37

More than a year has now passed since this work was completed, no new material having been added in the meantime, except a few loads of gravel screenings. The section between One Hundred and Fifty-ninth and One Hundred and Sixty-fifth Streets has had to stand an unusually heavy traffic, owing to the use of this stretch by trucks hauling material for the macadamizing of Boulevard Lafayette. The road shows a decided improvement, however, over its former condition, though it is somewhat cut up by ruts, and the dirt has broken through in a few places. The short strips covered with gravel screenings and feldspar screenings were not satisfactory. The feldspar screenings were quickly ground into dust or mud, while the gravel screenings resembled sand, and would not compact under traffic. The rest of the road has preserved its surface very well. The best sections are those covered with ¾-in. limestone and ¾-in. feldspar, materials which behaved especially well on hills. The coarse gravel furnishes a hard, firm surface, though not as smooth as that of the broken stone. It will be noticed that these three best sections cost between $1 000 and $1 280 per mile, or from 19 to 24 cents per running foot. In localities where stone and labor are cheaper, the cost would be considerably less. The writer wishes to bring out the point that on

Mr. Travell. this avenue, where the construction of an expensive pavement was entirely out of the question, and where the traffic consists chiefly of carriages and light delivery wagons, a roadway has been built which, as regards its present utility, is equal to a pavement costing ten times as much.

The limestone used was from the Tomkins Cove quarry. Its analysis may be found in the *Transactions*, Am. Soc. C. E., Vol. VIII, page 98. The stone referred to as feldspar came from Clinton Point, south of Poughkeepsie, N. Y. Its analysis is:

Silica	63.24	per cent.
Alumina and sesquioxide of iron	19.52	"
Lime	3.80	"
Magnesia	1.29	"
Carbonic acid	2.72	"
Alkalis, etc	9.43	"
	100.00	"

The average crushing strength of cubes of less than 2 ins. is 30 580 lbs. per square inch.

In the City of New York it is illegal to assess a piece of property for more than half its valuation at one time, so it is customary to allow a street or road to remain unpaved until the property increases in value sufficiently to stand the assessment for an expensive, permanent pavement. After this first pavement is laid, the city pays for all maintenance and repaving, but until this time the newly opened street or suburban road must struggle through a long period of growth with its surface quite impassable for at least three months of the year, and in poor condition during the remaining nine. This period of growth is, of all times, the most critical in the history of a locality —a time when, if at all, the city or county should extend its aid, and, by comparatively small expenditures, maintain such streets in a respectable condition until the permanent pavement is laid.

Such assistance would increase valuations so greatly that the increased taxation would pay for the improvement many times over, and, by thus fostering the growth of the locality, would hasten the time when the expense of a better improvement could well be afforded.

Mr. Cudworth. F. G. CUDWORTH, Esq. (by letter).—The writer, under the immediate direction of Edward P. North, M. Am. Soc. C. E., then Consulting Engineer of the Department of Public Works, New York City, had charge of the resurfacing of Seventh Avenue, between One Hundred and Tenth and One Hundred and Forty-fifth Streets, in the summer and fall of 1897. The area resurfaced was 81 675 sq. yds., and the average depth of material was, approximately, 3 ins.

The old roadbed was not broken up to a depth of 3 or 4 ins. by pick points in the wheels of the roller, but the surface, after a thorough

cleaning by scraping with stub wire brooms, was scored or pitted by hand labor with pickaxes, the pits being from 8 to 14 ins. apart. The old surface was in a satisfactorily compact condition, and the intention was to retain it as nearly intact as possible, and yet produce such roughness as would effectually prevent the sliding of the new material. On the roadbed thus prepared the new material was spread to the required depth, and the roller was passed over it twice without the addition of screenings or water. Then the entire surface was covered with trap-rock screenings containing about 20% of flour, and water was applied until the whole was thoroughly wet. The rolling was kept up constantly during the wetting, and, with the addition of more screenings, was continued until the new material was thoroughly consolidated, and enough screenings had been forced in to fill all voids in the coarse stone, and bring a thin mortar to the surface in front of the roller. Each block (80 ft. x 220 ft.) after being puddled in this manner was allowed to dry out for 10 or 12 hours, and was then covered with coarse screenings to the depth of ¼ in., and opened to traffic. Mr Cudworth.

To estimate the time and cost per square yard, accounts were carefully kept on eleven out of the thirty-five blocks, aggregating an area of 21 908.3 sq. yds. The time of roller work on this area was 464½ hours, giving as an average 47.15 sq. yds. finished per hour, with extremes of 38.4 and 65.1 sq. yds. per hour, or, in other words, the average time per 1 000 sq. yds. was 21.2 hours.

The cost of rolling per hour, allowing one-tenth of the watchman's time, was as follows:

Engineer	.30
Fireman	.15
Coal and oil	.40
Sprinkling	.30
Watchman	.15
Total	$1.30, or 2.765 cents per square yard.

The following amounts should be added to obtain the labor cost of resurfacing:

Rolling per square yard	$0.02765
Scraping and sweeping	0.02000
Picking old surface	0.01500
Spreading stone	0.02000
	$0.08265

or about 8¼ cents per square yard.

The amount of material paid for by the contractors and placed on the avenue was: 8 878 cu. yds. of 2-in. trap rock, and 4 763 cu. yds. of trap screenings. It will be noticed that the quantity of trap delivered overruns the assumed quantity on the road by about 30%, or, in other

Mr. Cudworth. words, 3.9 ins. of broken stone were spread on the road, and 2.1 ins. of screenings were added. There was no opportunity for settlement in this instance. The average thickness, however, may have been underestimated, though the contractor did not expect to put on over $2\frac{3}{4}$ ins. of compacted material, plus $\frac{1}{4}$ in. of screenings as a top dressing.

The work was finished in the latter part of December, 1897. No difficulty was experienced from the stone creeping in front of the roller, and no part of the road has at any time broken up. Seventh Avenue, in consequence of the repaving of Eighth Avenue and the delapidated pavement on Lenox (or Sixth) avenue, has had an unusual amount of heavy traffic since the completion of this work, and there is also probably more fast driving there than on any other street or road in the world.

Mr. McCann. T. H. McCann, M. Am. Soc. C. E. (by letter).—During the summer of 1890 the writer was called upon to take charge of the repairing of about 5 miles of telford and 2 miles of macadam roads in North Hudson County, N. J. These roads ran through five different towns, which supplied the necessary crushed stone, but the money to pay for the labor was subscribed by public-spirited citizens, one of whom, Col. E. A. Stevens, purchased for the work a 15-ton steam roller at a cost of $5 000.

The widths of the telford roads ranged from 20 to 30 ft., and the width of the macadam roads was 12 ft. The former had been constructed in a first-class manner thirteen years previous, but remained neglected up to the time of the repairing, and were worn down to the foundations; the latter were old, unscientifically constructed, stone roads in very bad condition.

The method of repairing was as follows: After the gutters, culverts and drains had been thoroughly cleaned out and graded, and the road surface cleaned of mud and slush, two courses of crushed trap stone, sufficient to bring the surface to grade, were applied to the foundation. The first course consisted of $2\frac{1}{2}$-in. stone, over which was sprinkled selected dry loam; upon this was spread $1\frac{1}{2}$-in. stone, and, after a lavish use of water supplied by the Water Company without charge, the whole was rolled until the binding came to the surface in puddle. This was swept off, trap screenings were spread on while the surface was still wet, and the road again thoroughly rolled, when it was ready for use. These repairs were done longitudinally, taking half the width of the road at a time. The plant consisted of a cart, a watering wagon and the steam roller. The working gang was composed of five men and a foreman. This force was engaged during four months at an expense of about 8 cents per square yard of road. This, with the cost of the stone furnished by the towns, made a total cost of about 20 cents per square yard.

From the foregoing and similar experiences, the writer is convinced

that this method of repairing is preferable to that of continuous patching in spots. In other words, it would seem to be better to allow a road to wear down a few inches, and then repair it as a whole. Mr. McCann.

As to the use of binder on new roads, the writer agrees with Mr. Owen that this is a matter of judgment, depending upon the locality, the kind of stone used, the nature of the traffic and the speed with which it is necessary to complete the work. The writer's experience with trap telford has been similar to Mr. Harrison's, *i. e.*, that excellent roads can be constructed without earth binder, but more time will be required.

One serious concern to the engineer is the construction of roads over swamps and peat bogs. After considerable experience in this line the writer has come to the following conclusions: If the line is over a soft, deep, wet and yielding swamp it may be the cheapest and best plan, provided timber is handy, to float the road on a raft made of long poles; a bottom course laid longitudinally, with broken joints, a second course laid transversely and covered with brush. The grade line should be the lowest permissible, and the filling free from deleterious matter, and the lightest obtainable, in order to reduce the weight as much as possible. Side-ditching should not be done along these roads, as this destroys the sustaining power of the peat; but drains should be made a considerable distance from the road. This also applies where embankments are constructed over swamps.

In filling in over peat bogs, it seldom happens that the embankment sinks to the hard underlying bottom, and in making preliminary estimates of the amount of filling necessary, it is very difficult to determine the correct settlement. It can only be said that on ordinary marshes it is not safe to assume a settlement of less than $1\frac{1}{2}$ ft. for each foot of height above the surface of the swamp for embankments whose grade lines are from 2 to 5 ft. above the peat surface.

As the writer stated in his discussion on "The Compressibility of Salt Marsh Under the Weight of Earth-Fill,"* there is no constant relation between the depth of the marsh to hard bottom, and the height of the fill above its surface. This relation is largely dependent on the nature of the swamp material. Observing the sinking of a sounding rod, and taking borings to hard bottom, will often aid in forming judgment in this matter. As stated before, it is desirable in constructing roads of this kind, to keep the grade line as low as possible. If the road is not to be used for railroad purposes, it need not be brought to extreme high-water line over a swamp where there are no currents. The inconvenience to the public from a foot or so of water on the road during the short period of an occasional extreme high water may not be of such a serious nature as to warrant the ex-

* *Transactions*, Am. Soc. C. E., Vol. xxxvii, p. 219.

Mr. McCann. pense of filling above that line. The first macadam covering should be of the thinnest, as there is sure to be some settlement which can be made good by the addition of stone from time to time, finally making a considerable thickness of metal on the road. Telford is not to be recommended on swamp roads.

A few years ago the writer constructed a trolley road over some tidal swamp land. It was a double-track line, each track being laid on the outer side of an old plank road. This old road had been made, many years before, by laying 3-in. planks transversely on the surface of the meadow. The grade line was about 2 ft. above the meadow, and about 2 ft. below occasional extreme high tide. The depth of the filling over the planks varied from 4 to 6 ft., and the depth to hard bottom was from 30 to 60 ft. In constructing the trolley line it was necessary to widen the road on either side about 3 ft. beyond the ends of the old planks, and to excavate 1 ft. for the subgrade. In doing this, great care was taken not to disturb or cut the peat surface, which was more or less firm, although in places not firm enough for a person to walk upon without sinking nearly to the knees. The filling was boiler-house cinders.

The trolley road was in operation over a year before any resurfacing was necessary, and then the settlement was found to be fairly uniform along the track. About once a year this road is submerged 18 ins. by the extreme high tide, thus making it necessary, for a few hours, to draw the cars by horses, the company having no authority to raise the grade.

In another embankment, built in 1897, over a tidal swamp on the shores of the Passaic River, a settlement of only 2 ins. was found after the road had been subjected to considerable heavy traffic for one year. This road is 80 ft. wide, with a 40-ft. carriageway. The grade line is an incline, varying from 4 to 20 ft. above the surface of the marsh. The swamp was a firm peat bog at the elevation of mean high water, but too soft for a person to walk upon without sinking nearly to the knees. The depth to hard bottom was 8 ft. below the bog surface. Borings into the embankment disclosed the fact that the compression of the marsh was nearly uniform; that is to say, the bottom of the 24-ft. fill was only 1 ft. lower than that of the 8-ft. fill. The preliminary estimate was 2 000 cu. yds. in excess of the quantity determined by the cross-section borings.

Mr. Burke. M. D. BURKE, M. Am. Soc. C. E. (by letter).—The writer does not expect to adduce anything new upon this subject, but wishes to offer a suggestion or two regarding the definitions of terms used by others in the discussion.

All agree that "thorough drainage" is necessary, yet no one has very clearly defined the term, or described the means by which it is to be secured. If the impression be conveyed that thorough drainage

is supplied when ditches or conduits are provided by which the rainfall is conducted from the roadway, the idea will only be correct in exceptional cases. In general, the soil supporting or underlying the roadway must be drained to give it the sustaining power to carry the traffic. If the soil underlying a level road becomes saturated, or if the soil water rises to the level of the roadway, no macadam or telford covering that can be placed upon it will sustain any considerable traffic. The drainage, therefore, to be thorough and efficient, must lower the water level 2 ft. or more below the roadbed. Mr. Burke.

Upon all gradients where the subsoil is not composed of sand or gravel, from which the ground water finds a convenient exit, the plane of the road will cut across lines or planes of pervious material, through or along which the water will reach the surface of the ground beneath the stone covering of the roadway, and destroy, not only the bearing power of the soil, but also the cohesion of the metal. If these currents be not intercepted (and they are seldom visible during the construction of the road), and the moisture conducted from them to the side drains, the drainage will not be thorough. Tile drains beneath the roadway are likely to become broken or clogged. The writer has found that trenches filled with porous material, such as coarse sand, sandy gravel or broken stone and sand, leading diagonally to the side drains are more reliable, and cost less, than any other form of subsoil drain that has come to his notice. The side drains described by Mr. McClintock will generally be efficient, but there are many localities where the diagonal subsoil drain, or a substitute for it, is necessary.

All clay soils that are to support macadam or telford roadways, after being properly drained and compacted by rolling, should be covered with a layer of coarse sand or fine gravel, and at frequent intervals this layer should extend to, and be efficiently connected with, the side drains. It is well to understand that the purpose of this material is to form a pervious stratum beneath the macadam, which will intercept the moisture escaping from the soil, and lead it to the side drains, rather than into the covering of stone, where it is likely to freeze and destroy the solidity of the road covering.

Upon any roadway covered with metal to a depth of 6 ins. or more, it is obvious that the sole use of the lower portion of the metal is to distribute the weight upon the soil beneath it. It follows, therefore, that the bottom or first course of stone can consist of any convenient material that is suited to that purpose. If the material be limestone, it must be free from shale, for upon exposure to the weather that soon becomes clay, which the frost uses as a disintegrating element in the roadway. If it be sandstone, it must have sufficient strength to resist crushing, and to prevent displacement by the harder fragments of stone overlying it when they are acted upon by the concussions and pressure of the traffic. Whatever the material may be, it is evident

Mr. Burke. that, where it is economical to do so, a compact limestone, a dense, hard sandstone, an inferior granite, or gneiss, will serve for this part of the road covering almost as well as the hardest trap rock. Whether it be placed in the form of a telford pavement, or broken into cubical fragments, the sides of which measure not more than 2 or 3 ins., will matter little, as the advantage will generally lie with that which becomes most compact under the roller, and can be placed in that form at the least cost. Under the pressure of the roller, the sand will be forced into the spaces between the stones, while if the sand be omitted, those spaces will be filled with clay, a condition adverse to drainage and conducive to destruction by the action of frost.

The wearing surface should be composed of the most refractory material, obtainable at reasonable cost, that can be united into a compact mass. The whin-stones, or trap rocks, and the harder granites, are the most desirable, but the limestones are most widely distributed and generally available. No valid reason can be assigned for making this coat more than 3 or 4 ins. in thickness. The fragments composing it should not be too small, as then they are more easily displaced, and consequently wear more rapidly. All crevices between them should be filled with finely crushed rock, be thoroughly worked in with steel brooms, while dry, and then be wetted and rolled until the surface is perfectly smooth and hard. No clay or other substance readily soluble in water should be used as a binder or for any other purpose. It is very pleasant to see the beautiful surface that can be produced so readily, during the summer and autumn months, by rolling the clay or loam with the crushed stone, but when the frost enters and not only picks the lock, but destroys the entire combination, the results are very discouraging. A sharp silicious sand is sometimes necessary to secure the requisite bond, and in the early repairs of the road it is probably the best material, if sparingly applied; but when dry it affords considerable dust. When the road is constructed of limestone, the screenings should be used without separating the dust from them, provided the stone was clean when it went to the crusher. Limestone screenings may be used with whinstone or granite, with which they will readily form a bond, and inasmuch as the crushing of the harder stones seldom affords sufficient fine material for completing the roadway, their use as a binder is quite common, and is generally satisfactory. In form, the fragments constituting the wearing surface should be as nearly cubical as can be obtained from crushers, and in size, they should not exceed 2 ins. or be less than $1\frac{1}{2}$ ins.

On a new road, the most troublesome factor is the steel-clad hoof of the roadster when traveling at high speed, as the blow which it strikes is likely to loosen fragments of rock, and a species of raveling out follows, if the defect be not remedied. No better remedy is

known to the writer than to keep the surface lightly covered with screenings, and have a trusty man with proper tools make frequent repairs of the incipient damages until they cease to appear. Mr. Burke.

The form to be given to the roadway should be such as to cause the travel to be distributed over its entire surface. The high crown will cause the traffic to follow the center of the roadway. No sensible horse will travel upon a side hill when he can as easily go on the top of it. The low flat curve will retain no water upon its surface, and will generally distribute the traffic effectually. There is no sensible method of maintaining roadways other than that of constant inspection and repairs. No road can be said to be in proper repair when it is thickly covered with dust or mud, so that cleaning and repairing should not be separated.

When the telford or bottom course of broken stone is omitted, and the soil is smoothed, drained and compacted, and has a thin layer of broken stone or gravel rolled directly into it, it becomes the thin road so interestingly described by Mr. Travell. Plainly, this form of road is adapted to the lanes and byways of the country, and the cost places it within the reach of the farmer, but its construction, and particularly its maintenance, requires appliances and skill which are not commonly used in the work done on country roads. Millions of dollars are annually expended in the United States in building and maintaining macadam and gravel streets and roads, but the methods used are in no way similar to those described in this discussion, and the results are not so beneficial as they might be, were the work done under skilled supervision.

HERMAN CONROW, Jun. Am. Soc. C. E. (by letter).—The question of the elasticity of a road seldom receives the consideration it deserves. During the summer of 1897 the writer had occasion to design and build a driveway for a large estate in Dutchess County, N. Y., where it was especially required that the road when finished should not be too hard. In order to accomplish this, screened gravel, varying in size from ¼ in. to ¾ in., was used for the top coat. This was evenly spread to a depth of 3 ins., on a 6-in. broken stone foundation, and then roughly wet and rolled. No binder was used, as enough soil adhered to the gravel to make it pack readily. The gravel was easily rolled solid, and, after drying, the surface was smooth and firm, making an ideal road for driving. Later experience proved that this road was capable of bearing considerable hauling without showing unusual signs of wear. Roads in the same neighborhood, made of a reddish iron ore, are most excellent for driving when in a dry condition, but become soft and sticky after a long-continued rain. For park purposes, with only light driving, such a road would give satisfaction because of its great elasticity. Mr. Conrow.

Along the banks of the upper Hudson are enormous deposits of

Mr. Conrow. gravel, occurring in pits of great depth, frequently 40 ft. or more. These deposits are composed almost entirely of smooth stones, with enough clayey material to hold them firmly together. When excavated, the stones adhere to each other in great lumps, and very much resemble concrete. The stones are of all sizes, from that of a pea to boulders as large as a man's head. The stone is soft and unfit for roads where heavy hauling occurs, but for driving roads it is an excellent material, making a firm road, yet very elastic.

In Burlington County, N. J., considerable difficulty has been experienced from building the roads too hard. In some instances a coating of gravel about 1 in. thick, spread over the macadam, has given the desired effect of reducing the extreme hardness. Upon one of the main roads leading into Camden, N. J., over which many hundreds of loads of vegetables are carted daily to Philadelphia, it is the custom for the drivers, as the trip is usually made at night, to sleep on the homeward journey, allowing the team to drive itself. It is a matter of interest that teams under such conditions are frequently seen traveling at the side of the road on dirt and sand—where the draught is much more difficult—in preference to drawing the wagon over the smooth but painfully hard telford. No doubt if horses had a voice in the matter, the question of the elasticity of roads would receive a great deal more attention than it does now.

Mr. LeConte. L. J. LeConte, M. Am. Soc. C. E. (by letter).—An engineer's opinions are based largely upon his personal experience and observation in his immediate neighborhood. The writer has given much attention to long-time tests of both telford and macadam roads made of stone from the same quarries.

His observations lead him to the conviction that a thin macadam pavement, 5 to 6 ins. thick or less, cannot be maintained under moderately heavy traffic, except when underlaid by unusually good natural soil. Hence, its economic application is quite limited, particularly where good stone is scarce. Time has shown conclusively that many of Macadam's "principles" are errors of judgment, *e. g.*, disregarding the nature of the soil, and laying the crushed rock directly upon the soft ground. Imperviousness to water was also particularly harped upon, a perfection difficult to obtain, in any case, and impossible to maintain. When the subsoil is not readily softened by water, or when the pavement is properly drained, permeability is not an objectionable feature.

The telford pavements of to-day meet the ordinary requirements of good roads better and are cheaper than macadam, and at the same time are more practical in every way in their adaptability to changing conditions. They are especially desirable on soft and wet ground.

In most States the highways pass through regions which can only supply second-class stone. In such cases the telford type is by far the

more economical, the expensive portion of the pavement being confined to the top 3 ins. of road metal proper—the main bulk below being composed of second or third class stone. Mr. LeConte.

One of the best object lessons, showing the intrinsic value of the telford pavement has been noted in connection with the study of the old telford roads at and near Oakland, Cal. Many of these are now twenty years old, and, as to repairs, have been shamefully neglected. Subjected to moderately heavy traffic, these roads have worn down well into the foundation course, presenting a surface nearly as rough as an old cobblestone pavement. These roads are now being repaired with remarkable success at a very small cost, a roadway for all practical purposes as good as new being obtained at the trifling expense of 18 to 27 cents per square yard. The roads are repaired as follows: *First*, the old subpavement is soaked with water so as to soften the foundation somewhat; *second*, a 15 to 20-ton steam roller is used to crush and mash down the rough surface, giving a proper crown and shape to the finished surface, and new material is added, if necessary; *third*, there is put on a uniform layer of from 2 to 3 ins. of first-class road metal composed of either trap or basalt. This layer is sprinkled and well rolled with the 15-ton roller, using as a binder some neighboring road detritus mixed with sharp sand.

Mr. McClintock's remarks relating to the economic importance of making use of the entire output from the crushers cannot be too strongly commended; but this economic principle should be extended to include the gross output of the quarries as well. Exclusive specifications cannot preclude the public from paying for the gross output of the quarries, whether it is made use of or not.

It is pleasing to note the substantial efforts being made in Massachusetts toward educating the public to fully appreciate the true value and importance of good roads. Taking a broad view of the question, it may be safely stated that bad roads cause farmers more financial loss annually than all the fluctuations in market values combined. It costs the farmers from 25 to 30 cents a ton per mile to get their crops to market, while the cost of hauling on good roads is only from 6 to 10 cents per mile. This unnecessary tax paid annually by the farmers because of bad roads is correctly estimated to be high up in the millions for each State in the Union.

W. C. Foster, Esq. (by letter).—In 1897, the Borough of Lodi, a small municipality in Bergen County, N. J., voted to macadamize and otherwise improve certain of its roads and streets. The appropriation for this work was very small in proportion to the territory it was wished to cover, being but $12 000. For this amount it was desired to grade and macadamize about 4½ miles of streets. It was, of course, impossible to do so with such a small sum, but there was sufficient to grade the 4½ miles and to macadamize about 3 miles. Mr. Foster.

Mr. Foster. The total excavation in the grading of the different streets was 11 668 cu. yds., which was used in filling up low places in the roads, and in widening the banks. The grades were kept as low as possible, the minimum being 0.8%, except in one or two instances, where too great damage to adjoining property would be done, or where the adoption of this grade would cause heavier work than the appropriation warranted. There were only one or two places where a lighter grade was used, and in these places there was not much water to be taken care of. These lengths were short—not over a few hundred feet long, and were located mainly at summits. Many of the streets were so narrow that nearly all the travel would be confined to the center, and, therefore, a heavy crown was adopted.

The material encountered in the subgrade varied greatly, and some of it made an exceedingly poor foundation for a macadam road. Loose, dead, shifting sand, sharp building sand, loam, hard, red clay, and a soft, wet, springy foundation were found, and these sometimes changed from one to the other in a very short distance. The funds being so limited, the foundation work was confined to excavating the worst places, and filling them in with the most suitable material at hand. Where shifting sand was encountered it was found that it could be consolidated sometimes by slightly dampening it with the watering cart, and sometimes by scattering a small quantity of broken stone over it. Some of the spongy material was not cut through by the roller, yet, after the roller had passed, it immediately rose from 1 to 2 ins. or nearly to its original level. After the subgrade had been formed, it was thoroughly rolled, until in fact the wagons used for hauling the stone could be driven over it without forming ruts.

Trap rock was then spread over the subgrade to a depth of 5.67 ins. for 4-in. macadam and 7.38 ins. for 6-in. macadam. The thickness of the stone was from 5½ to 6 ins. on 4-in. work, and from 7½ to 7½ ins. on 6-in. work. These thicknesses were calculated from the actual car measurements, and from the number of square yards laid. In order that these thicknesses might not be exceeded, levels at 50-ft. intervals were frequently taken on the subgrade, the loose stone, and the stone after rolling.

The trap rock was rolled dry until the required final thickness was reached. By making an experiment upon a piece of old macadamized road, the requisite thickness of loose stone was determined. The rolling of the stone was continued until a horse and wagon could be driven over it at a walk without pieces being kicked out, and until, in walking over it, the stones did not move under the foot.

The stone was broken and screened to such a size that it would pass over a hole 1⅛ ins. in diameter, and through one 2½ ins. in diameter. The writer thinks that larger sizes would have given better results, and in any future work he would use larger stone on the sur-

face, and if for economy it became necessary to use two sizes, would place the smaller on the bottom. Mr. Foster.

Upon the completion of the trap-rock work, fine limestone from Tomkins Cove, containing a large proportion of dust, was spread broadcast over the surface with shovels, watered and rolled until the interstices were completely filled and a very thin coating remained on the surface. As fast as the limestone disappeared, during the rolling, more was spread. During the process of rolling it was found necessary, as a rule, to keep the watering cart immediately in front of the roller, as otherwise the limestone would become sticky and pick up the road in patches. Rolling and watering were continued until the water flushed ahead of the roller, and the surface was covered with a liquid paste. If, during this process, the subgrade became so soft as to allow the wheel tracks of the roller to show in the stone work, work was stopped on that section, and the road-bed allowed to dry out for a day or so, and then the work was completed. In fact, it was found to greatly benefit the road to practically complete it, allow it to dry for a few days, and then go over it a few times with the watering cart and roller.

From observations made during the construction of these roads, it is the opinion of the writer that it would be of great benefit to them to sprinkle the surface immediately with a layer of sharp building sand from $\frac{1}{8}$ in. to $\frac{1}{4}$ in. thick, and to thoroughly water and roll it, in order to hasten the chemical action believed to take place.

Until the first winter has passed, it is important to prevent the limestone dust from being blown off the surface, and this can be easily accomplished by the occasional use of the sprinkling cart. After the first winter this precaution is unnecessary, and the surface may be swept, if desired, when the road will present a perfectly clean and smooth surface. Roads built in this manner do not soften in wet weather, or when the frost is coming out of the ground in the spring, nearly as much as those in which clay is used as a binder.

A piece of 4-in. road about 3 500 ft. long, built in the fall of 1897, did not ravel at all during the past summer, although it was not sprinkled. It presented a clean, smooth surface of trap. The travel over this section of road averages 130 wagons per day, from 7 A. M. to 6 P. M., some of them being very heavily loaded, and all having narrow tires.

A section of about 3 000 ft. of road, built in a defective manner by the contractor, was swept a few weeks after the contractor had abandoned it, as clean as it could be with steel street brooms, and opened to travel. Though a great deal of stone was hauled over it in very dry weather it did not ravel where it had been built with any reasonable respect for the specifications. The drippings from the watering cart in passing back and forth kept it in excellent condition.

Mr. Foster. Rain seems to have a different effect on the road than the water from the cart, and where the final rolling can be done during a rain-storm, the road becomes cemented up much more quickly.

The rolling was all done by a 12½-ton steam roller. The limestone dust is not washed off the road, but seems to form a sticky paste and remain on the surface until it is gradually blown off by the wind.

A short time after these roads were completed, considerable heavy hauling was done over a portion of them. For about five weeks at least two to four loads, of 5 to 7¼ tons each, were hauled over them daily, in wagons, having tires about 2 ins. broad. Not even a wheel-mark is to be seen on either the 4-in. or 6-in. work. The regular traffic over the 6-in. road averages 308 wagons per day, from 7 A. M. to 6 P. M., the loads running from a light wagon to about 4 tons.

The larger part of the work was completed during the summer of 1898. One short section, finished in 1897, seems to be better to-day than when it was first built.

The total sum available for all purposes—advertising, printing, bonds, engineering, first six-months' interest, and construction, was a little more than $12 700. The following table shows the amount and details of the work done. In addition to the items given in the table the new work was kept sprinkled for about two months after completion, and a balance of nearly $50 was left.

Excavation	11 668 cu. yds.
Ballast foundation	342 sq. yds.
4-in. macadam	18 430.8 sq. yds.
6-in. macadam	4 771.1 sq. yds.
Cast-iron pipe culverts	233.8 lin. ft.
Cobble paving	131.2 sq. yds.
Length of street graded	4.6 miles.
Length of street macadamized	3 miles.
Average thickness of loose trap rock on 4-in. work	5.67 ins.
Average thickness of loose trap rock on 6-in. work	7.38 "
Quantity of 4-in. macadam 1 cu. yd. of trap rock will lay	6.348 sq. yds.
Quantity of 6-in. macadam 1 cu. yd. of trap rock will lay	4.878 sq. yds.
Quantity of either 4-in. or 6-in. macadam 1 cu. yd. of limestone will complete	21 sq. yds.
Cost of rolling, per square yard of completed macadam	$0.0412
Average number of square yards of completed work done by roller per day	314 sq. yds.

JAMES OWEN, M. Am. Soc. C. E.—Mr. North has stated that he considered the practice in building deep roads, such as telford roads, was not to use packing, but that thin roads required packing. Unfortunately, the speaker builds the deep roads only, and always uses packing, so he is rather departing from the general practice. Mr. Owen.

The speaker wishes to suggest one thought connected with this view, *i. e.*, that the *desideratum* for the surface of a road, either of telford or macadam, is a homogeneous mass. There should be a mass consisting of stones so thoroughly wedged together that there could be no extraneous material between the particles; and they should have a mechanical bond. The speaker has seen a piece of macadam 10 x 8 ft. square taken up bodily, and it seemed to be as strong and as stiff as a piece of flagging. That is what should be accomplished, if possible. It is the practice of the speaker to use no more packing than is necessary to get this bond; that as the pressure by rolling or by wheel travel comes on the broken stone during wet weather, a large proportion of packing is squeezed to the top in that way. On that theory, of course, the more packing there is, the muddier will be the road. If in a trap-rock road the trap-rock screenings are used, unless there is sufficient cementing power in the trap rock (the Palisades trap contains a great deal more lime than the Orange Mountain trap), mechanical binding will not be secured. The interstices only will be filled up with this dust which cannot get out, and the ideal condition is not attained. This brings up the question of rolling. The speaker believes that the shape of the stones governs the amount of rolling required. Stones which break cubically take less rolling to get them down than stones which fracture with flat surfaces. As to the necessity of breaking up the surface of an old roadway to repair it, the speaker is glad to learn that Mr. Cudworth did not use the ruthless forks in a roller, but merely picked the road. An old road is resurfaced because it is too rough for travel, and the speaker's experience has been that a road in that condition needs no picking or breaking, and that very satisfactory results will be obtained by putting the stone on top of the old surface. He has repaired a great many miles of road in that way, with very satisfactory results.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 845.

THE LAUNHARDT FORMULA, AND RAILROAD BRIDGE SPECIFICATIONS.

BY HENRY B. SEAMAN, M. Am. Soc. C. E.

PRESENTED NOVEMBER 16TH, 1898.

In 1889 the author presented, in the form of a discussion on the paper on "American Railroad Bridges,"* by Theodore Cooper, M. Am. Soc. C. E., a review of the fatigue of metals, as demonstrated by the experiments of Wöhler, and of the application of the Launhardt formula to these results. As the paper treated the subject of bridges from a historical, rather than from a technical, standpoint, this portion of the discussion was withdrawn before publication, with the intention of presenting it later in the form of a separate paper. Owing to the pressure of other matters the subject was laid aside until the present time.

In bringing the matter again before the Society, it is proposed to note the most complete of Wöhler's experiments, to repeat the deductions which were made in 1889, and finally to place the matter in such form as to permit of its practical application to bridge construction. For this purpose a general specification for railroad bridges is presented herewith in order to elicit discussion.

The author's first practical training in bridge work was with the use of the Launhardt formula, as applied to bridge design upon the

* *Transactions*, Am. Soc. C. E., Vol. xxi, p. 1.

Pennsylvania Railroad by Joseph M. Wilson, M. Am. Soc. C. E. Later, in the service of bridge shops, he used the old method of dimensioning, as formerly outlined by Mr. Cooper and others; and when subsequently he returned to the use of the Launhardt formula, he recognized more fully the extra labor which its refinements and general application involved, although its results, as being universally applicable to long and short spans and to special conditions, were extremely gratifying where the item of time consumed in calculations was not a matter of importance. It was in the endeavor to simplify the application of this general principle, while yet retaining its technical value, that the original experiments, as published in *Engineering* in 1871, were more carefully examined.

The adoption of the Launhardt formula and the acceptance of the theory of "fatigue of metals" had met with the most strenuous opposition by those who had formulated or used the older methods of design. One of the most severe arraignments of this theory which came to the author's notice, was that of Mr. Cooper,* when he described the term "fatigue of metals" as "absurd and unscientific" and stated that "this result was totally at variance with the acceptance of the perfect elasticity of metals, as heretofore accepted," and although his later specifications provide for separate live and dead-load strains in parts of the structure, it is understood that the step was taken because of the general uncertainty in the effect of live load, rather than as any concession to the theory of "fatigue."

To the author's mind, the experiments of Wöhler not only demonstrated clearly the fatigue of metals, but also entirely destroyed the theory of their perfect elasticity, as formerly accepted. These experiments were made with that careful attention to details for which the German Engineers are justly famous, and though they involve such a variety of conditions as to render refined analysis difficult, yet they show a uniformity of results which leads to unquestionable deductions. These experiments have been of the greatest value in the study of the resistance of materials.

Wöhler's Experiments.

The material with which Wöhler experimented included wrought iron, mild steel, cast steel for axles, and spring steel, but the results obtained upon wrought iron and spring steel were so much more complete and satisfactory than the others that the present review may be confined to them.

* *Transactions*, Am. Soc. C. E., Vol. xvii, p. 181.

The experiments noted are as follows :

WROUGHT IRON.

Repeated Variations of Tensional Strains (of known amounts).

Maximum strain per square inch.	Minimum strain per square inch.	Number of applications of load before breaking.
51 360 lbs.	0	800
47 080 "	0	106 910
42 800 "	0	340 853
38 520 "	0	480 852
34 240 "	0	10 141 645
47 080 "	21 400 lbs.	2 373 424
*47 080 "	25 680 "	4 000 000*

*And not broken.

Repeated Application of Transverse Strain (varying between known amounts).

Maximum strain per square inch.	Minimum strain per square inch.	Number of applications of load before breaking.
58 850 lbs.	0	169 750
53 500 "	0	420 000
48 150 "	0	481 950
42 800 "	0	1 320 000
38 520 "	0	4 035 400
*34 240 "	0	3 420 000*
*32 100 "	0	48 200 000*

*And not broken.

Rotating Bars, Fixed at One End, and Loaded at the Other (giving alternately equal strains in opposite directions).

Test pieces with filleted shoulders.

Strain per square inch.	Number of applications of load before breaking.
34 240 lbs.	56 430
32 100 "	99 000
29 960 "	183 145
27 820 "	479 490
25 680 "	909 810
23 540 "	3 632 588
21 400 "	4 917 992
19 260 "	19 186 791
*17 120 "	132 250 000*

*And not broken.

Test piece with square shoulder.

Strain per square inch.	Number of applications of load before breaking.
19 260 lbs.	1 603 570
19 260 "	2 063 760
*17 120 "	14 695 000*

*And not broken.

From these experiments Wöhler recommends the following as the safe limits of strain per square inch:

Maximum strain.	Minimum strain.
— 17 120 lbs.	+ 17 120 lbs.
— 35 310 "	0
— 47 080 "	— 25 680 lbs.

UNTEMPERED CAST STEEL FOR SPRINGS.

(Material furnished by Krupp.)

Repeated Application of Transverse Strains (varying between known limits).

(SET No. 1.)

Maximum strain per square inch.	Minimum strain per square inch.	Number of applications of load before breaking.
107 700 lbs.	0	39 950
96 300 "	0	72 450
85 600 "	0	132 650
85 600 "	0	117 000
74 900 "	0	197 400
64 200 "	0	468 200
*53 500 "	0	40 600 000*

*And not broken.

(SET No. 2.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
107 000 lbs.	17 762 lbs.	89 238 lbs.	62 000
107 000 "	35 631 "	71 369 "	149 800
107 000 "	53 500 "	53 500 "	400 050
107 000 "	62 381 "	44 619 "	376 700
*107 000 "	70 620 "	36 380 "	19 673 300*

*And not broken.

(SET No. 3.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
96 300 lbs.	21 400 lbs.	74 900 lbs.	81 200
96 300 "	32 100 "	64 200 "	156 200
96 300 "	42 800 "	53 500 "	225 300
96 300 "	53 500 "	42 800 "	1 238 900
96 300 "	53 500 "	42 800 "	300 900
*96 300 "	64 200 "	32 100 "	33 600 000*

*And not broken.

(SET No. 4.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
85 600 lbs.	10 700 lbs.	74 900 lbs.	99 700
85 600 "	21 400 "	64 200 "	176 300
85 600 "	32 100 "	53 500 "	619 600
85 600 "	32 100 "	53 500 "	2 135 670
*85 600 "	42 800 "	42 800 "	35 800 000*
*85 600 "	42 800 "	42 800 "	38 000 000*
*85 600 "	59 920 "	25 680 "	36 000 000*

*And not broken.

(SET No. 5.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
74 900 lbs,	10 700 lbs.	64 200 lbs.	286 100
74 900 "	21 400 "	53 500 "	701 800
*74 900 "	26 750 "	48 150 "	36 600 000*
*74 900 "	31 100 "	42 800 "	31 150 000*

*And not broken.

From these experiments Wöhler made the following recommendations as the safe limits of strain:

Maximum strain.	Minimum strain.
— 53 500 lbs.	0
— 74 900 "	— 26 750 lbs.
— 85 600 "	— 42 800 "
— 96 300 "	— 64 200 "

These experiments show that the material may be broken by the repeated application of a strain much less than that which would be required to produce rupture by a single application. The five sets of experiments on cast steel each show that as the amount of the strain which is repeatedly applied is decreased, the number of applications required to produce rupture is increased, until a strain is reached which will not produce rupture after an infinite number of applications. Sets Nos. 2 to 5, inclusive, also show in each case that the material may sustain a given maximum strain, but that fracture may be produced by repeatedly removing a portion of this strain. It is evident, therefore, that this change of strain, with the resulting fatigue produced by it, is sufficient to cause rupture under the conditions noted. Could there be any more convincing demonstration that it was fatigue, and fatigue only, which produced the varying results in each set? The opposition to the "fatigue of metals," in the face of these experiments, would seem to be an objection to the use of a word rather than to the acceptance of an idea.

Referring to the tabulated results of the tests on wrought iron, it is not apparent where Wöhler derives the limit which he recommends for all live load in one direction, *i. e.*, — 35 310 lbs. — 0 lbs., since the experiments, both with transverse and with torsional strains, would indicate — 34 240 lbs. — 0 lbs. If the latter limit were substituted for that recommended by Wöhler, the allowable strains would be modified as follows:

— 17 120 lbs.	+ 17 120 lbs.
— 34 240 "	0 "
— 47 080 "	— 25 680 "

Although these results are derived from experiments with different kinds of strain and are too brief to permit of the formulation of any definite theory, there seems ample basis for a general theory of work, *i. e.*, that the material is capable of doing a certain amount of work (derived possibly from the treatment, or work, which it receives in manufacture), and when that amount of work is exceeded, the material becomes fatigued, and its tenacity finally destroyed. Under these circumstances the destructive effect of live strain may be considered as bearing a certain relation to that of dead strain, as suggested by Gerber, and in the experiments on wrought iron just cited, that effect would seem to be about in the ratio of 2 to 1 for all cases, thus:

Wöhler's Tests.		Live strain. (Maximum — Minimum.)	Dead strain.	Equivalent dead strain. (Dead + 2 Live.)
Maximum.	Minimum.			
—17 120 lbs.	+17 120 lbs.	34 240 lbs.	0	68 480 lbs.
—34 240 "	0 "	34 240 "	0	68 480 "
—47 080 "	—25 680 "	21 400 "	25 680	68 480 "

The exact coincidence of these results—as shown in the column of equivalent dead strain—is striking, and although they are too incomplete to be considered as the demonstration of an exact law, yet they may be regarded as confirming the general theory of work.

The experiments on spring steel are equally interesting, and are the most complete and satisfactory of any of those made by Wöhler. They demonstrate conclusively the fatigue of metals; and the fact that they were made upon a homogeneous material makes them of increased value, since any flaw in the material is quickly detected in the results and the misleading test thrown out.

Set No. 1 of these experiments is merely a repetition of the action shown in the experiments on wrought iron. The applied load was gradually diminished for each successive experiment, until one was reached which would not produce rupture after an indefinite number of applications. In the second and following sets a method of procedure was adopted which removed all doubts as to the cause of rupture, as has been already described.

It would hardly seem necessary, in view of these experiments, to go further into the demonstration of the fatigue of metals, but in this connection the results of Bauschinger are so pertinent that they should not be overlooked. By the use of a delicate micrometer he ascertained that, at the very beginning of loading, a slight permanent set was noticeable. How far the element of time affected the result is difficult to determine, since there appeared to be a tendency in the material to return to its former state of rest for days, and sometimes weeks, after the load was removed. The fact of this slight set is probably the true explanation of the results obtained by Wöhler, and it has so modified former ideas of the perfect elasticity of metals, that the term must be abandoned or a new definition sought. Bauschinger himself proposed to use the more correct expression, "Limit of Uniform Elongation," and many engineers have preferred the concise term, "Yield Point," but others still adhere to the old name, "Elastic Limit," trusting to a considerate profession to give it the revised interpretation.

The Launhardt Formula.

These experiments on spring steel are so much more systematic and complete than any others made, that they have been generally regarded as presenting the most accurate information obtainable upon the subject of fatigue of metals, and it was in the endeavor to deduce an expression which would indicate the effect of this fatigue, as compared with that of permanent strain, that Launhardt outlined his formula.

The derivation of the formula* shows that, although it was sought by a rational method of procedure, it contained an arbitrary step which made the value of the formula entirely dependent upon the agreement of its results with those of experiment.

The formula, as outlined by Launhardt, is

$$a = u\left(1 + \frac{t-u}{u}\,\frac{\text{Min. Strain}}{\text{Max. Strain}}\right)$$

Where a = safe working strength,
u = safe strength for variable load,
t = safe strength for permanent load.

Since the value of this formula depends upon the agreement with experiment; and since the general theory of work suggested will depend upon a similar comparison, it is interesting to note what may be shown in this respect.

In comparing the results of the formula with those of experiment, the value of u was taken from set No. 1 as 53 500 lbs. (500 centners), but the value of t, not having been found in any of the duration experiments made on spring steel, was assumed, and a value of 117 700 lbs. (1 100 centners) was taken, which would bring, by the formula, the intermediate results of sets Nos. 3 and 4 in conformity with those of the tests. Had any other value of t been taken it is evident that the intermediate results would have varied accordingly.

The following table is reduced and corrected from the translation and shows to what extent the results of the formula agree with the experiments made:

Set.	u of Formula. No. 1.	No. 5.	No. 4.	No. 3.	t of Formula.
By experiment (Minimum.)	0	26 750	42 800	64 200	not found.
By experiment (Maximum.) a	53 500	74 900	85 600	96 300	.....
By formula for a	Assumed 53 500	76 430	85 600	96 250	Assumed 117 700

* Weyrauch, "Structures of Iron and Steel." Trans. by Du Bois.

It will be noted that in two instances (sets Nos. 4 and 3) the formula gives results corresponding to those of experiment, but since any other value of t than that assumed would have given different results, the comparison can hardly be considered a confirmation of the formula.

Let the general theory of work, as suggested in the experiments on wrought iron, be now applied to these same experiments on spring steel. It has been seen already how strikingly the results agree in iron—too precisely, perhaps, for absolute confidence, since the material is of such variable character—but in the tests with steel the variable results which would naturally occur can be followed more closely. Using the same ratio of 2 to 1 in these results, as was used for wrought iron, the following table is derived :

Set.	Wöhler's Tests.		Live Strain (Maximum — Minimum.)	Dead Strain (Minimum.)	Equivalent Dead Strain (Dead + 2 Live.)
	Maximum.	Minimum.			
No. 1.	53 500 lbs.	0	53 700 lbs.	0	107 000 lbs.
" 5.	74 900 "	26 750 lbs.	48 150 "	26 750	123 050 "
" 4.	85 600 "	42 800 "	42 800 "	42 800	128 400 "
" 3.	96 300 "	64 200 "	32 100 "	64 200	128 400 "

The column of Equivalent Dead Strains in this table shows that the results of sets Nos. 4 and 3 again agree, as they did with the Launhardt formula, but that sets Nos. 1 and 5 fall below them. By examining the tests of set No. 1, it will be noticed that 64 200 lbs. was applied 468 200 times before fracture, and that there was no test between 64 200 lbs. and 53 500 lbs. There is no assurance, therefore, that a weight exceeding 53 500 lbs., and perhaps nearly, or quite, equalling 64 200 lbs., could not have been applied indefinitely. Had 64 200 lbs. sustained the tests satisfactorily, the equivalent dead strain for this set would have been 128 400 lbs., and would have exactly agreed with sets Nos. 4 and 3.

Similarly, in the second test of set No. 5, a maximum load of 74 900 lbs., and minimum of 21 400 lbs., was applied 701 800 times before fracture, and no experiment was made between these results and those of the third test, recommended by Wöhler, of maximum 74 900 lbs. and minimum 26 750 lbs. Had something near the second test of this set been found satisfactory, instead of the third test as recommended, the equivalent dead strain would have been modified, as in set No. 1, to more nearly equal 128 400 lbs., as in the other sets.

Although these deductions are by no means conclusive, they are extremely interesting, and when the results on wrought iron are also considered, this ratio receives much more confirmation than does the Launhardt formula. The "fatigue of metals" is clearly established, a theory of work seems also well founded, and an approximate ratio of 2 to 1 for the relative effect of live and dead strains appears to be sustained by experiment. For the purpose of placing the subject before the Society in a practical form, it has been embodied in the specifications given in the Appendix.

Allowable Strains.—In determining the allowable strains for proportioning parts, a permissible dead strain is selected, and all other conditions are reduced to this basis. Dead and live strains are calculated separately as usual, and proper additions made to the live load for effect of impact. The strains which are resisted by columns may be properly provided for by the column formula, the whole reduced to equivalent dead strain and the required sections calculated in the usual way. This method is applicable to all conditions and locations of members in the structure, and avoids special provisions for local parts as was the common practice in the old method of dimensioning.

An allowable unit strain of 18 000 lbs. per square inch for dead strain may be selected as corresponding to the present practice in steel, though, where desired, this may be increased to 20 000 lbs. without modifying the general method of dimensioning outlined. The experiments of Bauschinger indicate that tensile and compressive strains may be treated alike, except that it is necessary to consider their algebraic signs and the effect of flexure on the outer fiber of long compression members.

In considering the relative effects of live and dead strains, it has been shown that Wöhler's experiments indicate a ratio of 2 to 1, but they were made without intervals of rest and without impact or any other increase of live strain beyond that noted in the tables. In outlining general specifications provision must be made for extreme conditions. In the elevated railways of large cities, or in the large railway terminals, where the passage of trains is almost incessant, there is little rest for the structure; while the vibrations of all rapidly passing loads, together with unbalanced drivers, flat car-wheels and defective track, will increase the live strains very considerably in addition to

the impact due to sudden application. When it is also remembered that according to Bauschinger it takes days, and perhaps weeks, for the material to fully recover from an applied strain, the ratio of 2 to 1 for dead and live strains does not seem excessive.

Having adopted the allowable dead strain per square inch, and accepted the ratio of 2 to 1 for the relative effect of live and dead strains, it is yet necessary to provide for the effect of impact and for the increase of compressive strains due to flexure of columns. The column formula in general use for compression is directly applicable to the proposed method of dimensioning, and may be adopted without change, except that the numerator becomes 18 000 lbs., according to the allowable dead unit strain. For alternate strains, however, the increase of compressive strain on the outer fiber due to flexure of column is found first, and then the compressive strain thus increased is added to the tensile strain, in order to find the total live strain. This increased strain due to flexure may be found by using the reciprocal of the column formula.*

In making provision for impact, there is neither a rational formula nor any complete system of experiments upon which to base a specification, and the practice varies from making no provision at all to making an allowance of 100% for floor system and adjacent connections. Special allowable strains for certain members, such as floor-beam hangers, have been provided in some instances, but it is desirable to avoid all "special cases" in a general specification, and to so frame the general clause as to apply to the elements upon which the special conditions are based. The ratio of 2 to 1 for the allowable dead and live strains may be considered as providing sufficiently for impact on long-span structures not in continuous use. For short spans of 100 ft. or less, additional provision seems necessary on account of the sudden application of the load. To meet this requirement the author would suggest the use of the table contained in the specifications until experiment furnishes authoritative data.

* This is readily explained as follows: The general formula is, $p = P\left(\frac{1}{1+\frac{l^2}{a\,r^2}}\right)$ in which P is the greatest strain allowable on any fiber of the column, and p is the reduced direct strain which may be applied at the ends of the column to produce the fiber strain P by flexure. If, therefore, we have given, the direct strain to be applied to the ends of the column and desire to ascertain the resulting increase of strain on the outer fiber, the formula becomes,

$$P = p\left(1+\frac{l^2}{a\,r^2}\right)$$

Those familiar with the derivation of the formula will recognize this as retracing one step in its original formation.

Specifications.—The bridge specifications presented as an appendix to this paper, it is believed, correspond to the best practice of the present time. The method of dimensioning conforms more closely to the results of Wöhler's experiments than does the Launhardt formula, and at the same time is direct and simple in its application. The clauses have been made rather full in detail, since it is an easy matter to omit those which are not desired, and it is hoped that the discussion will be so complete that it will enable a fairly uniform general practice to be established, and to that extent to anticipate the work of a committee upon this subject.

The specifications have been written from the standpoint of the railroad company rather than from that of the manufacturer, since it is the company's interests which they are to protect, and since, also, the railroad is later to maintain the structure. It is advisable for every railroad system to have a corps of bridge engineers to design and maintain this class of work, and, when the size of a road does not justify such an organization, several roads may unite in its employment. Railroad engineers would welcome the assistance of a specialist, provided that in so doing they did not jeopardize their own positions.

The specifications provide, either for plans furnished by the railroad company, or for plans furnished by the manufacturer and approved by the company. In either case all original drawings should become the property of the railroad, and for convenience in filing, a standard size of sheet should be adopted.

The extreme refinements necessary for the design of continuous draw-bridges and the impracticability of obtaining the theoretical reactions assumed in calculations has led, in draw-bridge design, to the use of two spans, which may be lifted and revolved, but which, in service, have the direct bearings and reactions of a single span. The use of continuous girders is therefore restricted to those cases where the device of separate spans may be impossible.

It is preferable that the ties, guard-rails, etc., of the floor system be constructed by the railroad company, as men are usually employed for purposes of maintenance and the work will be better and more economically performed by them than by contract, but where this cannot be done the specifications describe the work required of the contractor.

In discussing the paper on "Railway Bridge Designing,"* the author fully reviewed the subject of engine loading. The typical engine here specified corresponds closely to actual conditions. It represents the heaviest engine and train now in general use, though exceptional cases exist which are somewhat heavier, and ore cars are now in service which weigh 4 800 lbs. per lineal foot of track. The loading may be varied to suit different railroads, by increasing, or diminishing, all concentrations simultaneously in the same ratio, as has been proposed by Mr. Theodore Cooper. It is in no case advisable, however, to vary the loading more than 25% either way from that here specified.

The allowable dead strain specified is 18 000 lbs. per square inch, with the usual reduction for compression members. The column formula of Gordon, as modified by Rankine, is used and needs no explanation or endorsement. The impracticability of obtaining columns with fixed ends, in a bridge structure, is the reason for the adoption of only one column formula. In the case of alternate strains the column formula is replaced by its reciprocal,

$$P = p\left(1 + \frac{l^2}{a\,r^2}\right)$$

as already explained, but alternate strains are to be avoided wherever possible.

Where long unbraced compression flanges are used, the allowable strain is reduced by the Rankine formula,

$$C = \frac{18\,000}{1 + \frac{l^2}{5\,000\,w^2}}$$

but it is so customary to brace top flanges at intervals of about twelve diameters that this formula is rarely used.

The shearing and bearing values of rivets and pins have been reduced to the same basis of allowable dead strain; shear being three-fourths of the allowable unit strain for direct tension, and bearing being one and one-half times the same unit. If the basis of 18 000 lbs. is modified, these values should be correspondingly changed. The author is not aware of any specification which has made this distinction between live and dead strains in proportioning details, except those which adopt the Launhardt formula throughout. The details are the most important part of a structure, and those experienced in

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 227.

maintenance will testify that it is the live strain which causes loose rivets and which wears out the bridge. To provide for dead and live strains in the main members, while making no such provision for the details, is to build a scientific structure upon a crude foundation.

In outlining the details of design the endeavor has been made to cover every material point in first-class work.

The general tendency of recent practice is toward the adoption of soft steel for structural work. Drilling from the solid is obsolete, punching is almost universal, and perfect reaming is only performed when particularly required by the specifications. The injury to steel from rough punching and the danger resulting from flaws increase with the hardness of the steel, and for this reason the soft steel will doubtless continue in preference. The recommendations of the "Committee on Uniform Methods of Testing Materials, etc.," have been generally adopted, except that a milder steel than was there suggested is here specified. Where the rivet holes are punched $\frac{1}{8}$ in. small, and afterward drilled out to full diameter, it might be advisable to use steel of ultimate strength, 60 000 lbs. $\pm$ 4 000 lbs., and elongation 25 per cent. In such case the base for allowable dead strain could be increased to 20 000 lbs. per square inch.

The subject of painting is a matter of controversy. The first essential of a good paint is pure linseed oil; yet it is rarely obtained, and an adulteration is almost impossible to detect. Objection has been made to the practice of giving the steel a coat of oil in the shops, because of the necessity of having the oil thoroughly dry before the paint is applied, but the many advantages of oil for purposes of inspection, together with the fact that the oil usually has several weeks in which to dry before the coat of paint is applied, make it very desirable as a shop coat. For the subsequent coats the author prefers red lead, where the structure is to be exposed to the weather, and a carbon paint where exposed to locomotive fumes. In either case it is important that pure oil should be used. With red lead the difficulties of mixing and applying the paint make it important that only men experienced in this work should be employed.

Upon the matter of inspection the author has little to say. There is no element in bridge construction more important, if properly done, yet none so likely to be neglected. Few appreciate the importance of perfect mechanical work until they maintain the structures they build.

APPENDIX A.

SPECIFICATIONS FOR STEEL RAILROAD BRIDGES.

Proposals and Drawings.

Each bidder shall submit with his proposal complete strain sheets, showing loads assumed in calculations and the resulting strains, and the sections used. The strains from each kind of load shall be shown separately, and the dead load assumed shall not be less than that of the finished structure. At the same time, each bidder shall submit general plans of the structure, and such details as will show clearly the construction of all members and connections. In case of draw bridges, all machinery shall be similarly shown.

All drawings furnished by the railroad company, whether of general location or of details of construction, shall be strictly followed.

Upon the award of the contract and before work is commenced, a complete set of working drawings, in duplicate, including strain sheets and general drawings previously mentioned, shall be submitted to the railroad company for approval. All material ordered or work done before the drawings are approved shall be at the risk of the contractor.

All drawings shall be made on the dull side of tracing cloth, and of a uniform size of 24 x 36 ins. After the work is completed, these drawings, in good condition, shall become the property of the railroad company for file. The contractor may retain such prints or copies of them as he may desire for record.

General Provisions.

The structure shall be wholly of rolled steel and wrought iron. A type of truss shall be used in which the strains may be readily calculated and which subjects no member to alternate strains. Continuous girders will be allowed only in case of upper chords carrying floors, and in special cases of draw bridges.

Double-track, through-truss bridges, shall have only two trusses, and four-track bridges only three trusses, unless otherwise specified. In the case of plate girders, special provisions shall be made for spreading the tracks when necessary.

The distance from center to center of double track is 12 ft. 6 ins.

All through-bridges, on tangents, shall have a clear opening as shown on the clearance diagram, Fig. 1.

The width shall be proportionately increased for two or more tracks. On curves, the height and width shall be increased as required by curvature and elevation of rail.

All bridges shall be provided with steel cross-girders and stringers, except in the case of deck bridges, in which the ties may rest on the top chord, and be increased in depth to carry a single track.

Stringers and deck-plate girders shall not be spaced further apart, between centers, than their depth, with a minimum limit of 5 ft.

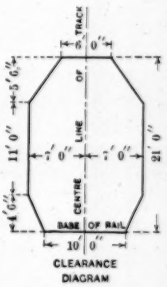


FIG. 1.

The wooden floor of ties and guard rails will be furnished and put on by the railroad company, unless otherwise specially provided by contract. The cross-ties shall be of long-leafed yellow pine, 8 ins. square and 9 ft. long, spaced 6 ins. apart in the clear and notched ½ in. over the stringer. In cases where the stringers are spaced more than 7 ft. apart, the depth of the tie shall be increased so that the strain on the outer fiber does not exceed 1 000 lbs. per square inch, considering the weight of a single driver as being carried by two ties.

Guard rails of long-leafed yellow pine, 6 x 8 ins. shall be placed 3 ft. 9 ins. in the clear, from the center of the track. They shall be notched ½ in. over the cross-ties and spliced with a horizontal half and half joint, 6 ins. long, over a tie. They shall be bolted to the cross-tie at the splice, to the cross-tie next adjacent to the splice, and to every fourth tie between, by ¾-in. bolts. To all other ties they shall be fastened by ¾-in. square spikes. Inside rail-guards shall be placed at a distance of 7 ins. in the clear inside the track rail, and shall extend back over the approach to a point not less than 50 ft. from the back wall.

LOADS.

The structure shall be designed to resist the strains from the following loads:

The dead load shall consist of the entire weight of the structure, properly distributed at the various panel points.

The weight of rails, guard rails, splices and bolts shall be estimated at 175 lbs. per lineal foot of track; ties of standard dimensions at 225 lbs. per lineal foot of track, and special ties at 4½ lbs. per foot B. M.

The live load shall be the moving load, with impact. The moving load shall consist of two typical consolidated engines, followed by a train, and distributed as shown in the diagram, Fig. 2.

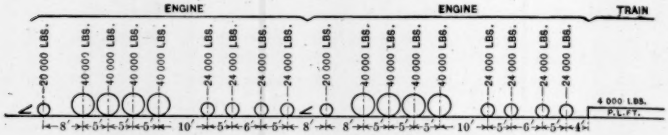


FIG. 2.

There shall be as many trains as tracks, each placed in such position as to produce the maximum strains in the structure.

The impact will be the increase of live strain in any member, due to the sudden application of the moving load, and shall be provided for according to the following table, by interpolation :

L (ft.)	10	20	30	40	50	60	70	80
I (%)	30	20	14	10	7	4	2	1

Where L = Distance in feet through which the moving load must pass to produce the given strain.

I = Percentage of increase of moving load.

When the bridge is on a curve, the lateral bracing and vertical trusses shall be proportioned to resist the centrifugal force due to as many trains as there are tracks, moving in the same direction, at the rate of 60 ft. per second.

Provision shall be made for the sudden starting or stopping of trains, estimating the coefficient of sliding friction at 15 per cent.

Provision shall be made for wind pressure, acting in either direction horizontally : 1st, of 30 lbs. per square foot of the surface of all trusses and the floor, as seen in elevation, considering the ties solid area ; in addition to a train 10 ft. high, beginning 2 ft. 6 ins. above base of rail, and moving across the bridge ; 2d, of 50 lbs. per square foot of exposed surface as specified above, without the train. The wind forces shall be properly distributed between the upper and lower chords.

In the case of plate girders, only one girder need be considered.

Provision shall be made for a variation of temperature of 150° Fahrenheit.

ALLOWABLE STRAINS.

The strains from the dead and the live load shall be calculated separately. The live strain will be the total variation in strain produced by the live load. The dead strain will be the minimum strain to which a member is subjected.

The allowable live strain per square inch shall not exceed one-half of that permissible for dead strain.

The allowable dead strain per square inch for rolled steel shall not exceed 18 000 pounds.

For columns subject to direct compression only, the allowable working strain of 18 000 lbs. per square inch shall be reduced, in proportion to the ratio of length to least radius of gyration, by the following formula :

$$p = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{18\,000\, r^2}}$$

Where p = allowable dead strain per square inch, in pounds,
l = length of column, in inches,
r = least radius of gyration of cross section, in inches.

For columns subject to alternate strains of tension and compression, the compressive strain shall be increased, to provide for the increase of strain due to flexure, in proportion to the ratio of the length to the least radius of gyration, by the formula :

$$P = p\left(1 + \frac{l^2}{18\,000\, r^2}\right)$$

Where p = direct compression in member,
P = increased strain due to length of column,
l and r = length and least radius of gyration of cross-section, in inches.

The total compressive strain thus found shall be added to the tensile strain, and the resulting strains per square inch shall not exceed those specified for live strains.

In case of compression flanges of beams and girders, the allowable working strains per square inch of such flanges for dead strain shall be computed by the formula :

$$C = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{5\,000\, w^2}}$$

Where c = allowable compressive strain per square inch,
l = unsupported length of compressed flange, in inches,
w = width of flange, in inches.

The shearing on pins, rivets, and bolts shall not exceed for dead strain, 13 500 lbs. per square inch of cross-section. Where tension on rivets is unavoidable, it shall not exceed one-half the limit allowed for direct shear. When a force is oblique, the components of direct tension and of direct shear shall be considered separately and the results combined.

The bearing pressure on pins, rivets, and bolts shall not exceed for dead strain 27 000 lbs. per square inch (diameter × thickness).

In cases of field rivets, driven by hand, an excess of 25% shall be allowed.

Wind pressure, centrifugal force, and sliding friction, either separately or combined, shall not produce greater strain per square inch than that allowed for dead strain.

The pressure, in pounds per lineal inch of roller, shall not exceed $500 \sqrt{d}$ for total load, (d = diameter, in inches).

Bed plates on masonry shall be so proportioned that the greatest pressure on the masonry does not exceed 300 lbs. per square inch.

Details of Design.

The assumed spans for calculation shall be as follows :

For pin-connected trusses—distance between centers of end pins.
" riveted girders " " " " bearing.
" cross-girders " " " " trusses.
" track stringers " " " of cross-girders.
" cross-ties " " " of track stringers.

The assumed depth for calculation shall be :

For pin-connected trusses—distance between centers of chord pins.
" riveted girders " " " " gravity of flanges (not to exceed the distance out to out of angles).

The whole of the wind force due to the train and floor, and one-half of the truss, shall be considered as acting on the lateral system of the loaded chord, and that due to one-half the truss only, on the lateral system of the unloaded chord.

In the case of deck bridges and very heavy curves, some of the centrifugal force may be transferred to the lower lateral system, in which case the truss shall be duly strengthened. The end portal bracing in through bridges must be of sufficient strength to transfer the accumulated wind strains from the upper lateral system to the end posts, and the end sway-bracing in deck bridges shall carry the whole of the accumulated wind and centrifugal forces from the loaded chord to the abutment.

Each main panel of deck bridges shall be provided with intermediate sway-bracing rods, of a sufficient section to carry one-half the maximum increment due to wind on train, and to centrifugal force. Through bridges shall be provided with post brackets at the intermediate panel points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure, or, when the height of top chord exceeds 25 ft. above base of rail, an overhead system of sway bracing shall be used.

Tension at the windward column of trestle piers shall be avoided if possible, and in any case approved anchor bolts well secured to the masonry shall be used.

The struts shall be proportioned to withstand their component of strain. No reduction shall be made from chord section on account of the material in the lateral system.

All parts shall be so designed that the strains coming upon them may be definitely calculated. The center line of resistance of a member will be along its neutral axis, and connections shall be so designed as to avoid bending, twisting or unequal tearing of the member or its details. The line of strain shall pass centrally through any cluster of rivets which resist it, and where angles or plates are otherwise than so connected, proper provision shall be made for the moments and secondary strains produced. Details will be so designed as to give free access for inspection and painting, and water pockets shall be avoided. In every case the connection of details shall be of greater strength than the member itself.

All members which are subject to direct strains, in addition to bending moments, shall be so proportioned that the algebraic sum of the strains coming upon them shall not exceed the specified allowable strains, properly reduced in case of columns. In continuous upper chords of deck bridges, carrying the floor, the strains due to the live load shall be computed from a bending moment equal to $\frac{3}{4}$ of the maximum moment produced by the engine on a span equal to a panel length considered as a simple beam.

The strain on the outer fiber of solid shapes shall be computed from the moment of inertia of the section.

No allowance shall be made for the web in calculating the flange section of plate girders.

In every case at least one upper flange plate on plate girders shall extend from end to end of the girder, and any additional plates used to make up the flange section shall be made of such lengths as to allow at least two rows of rivets of the regular pitch being placed at each end of the plate beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of the plates to transmit their value before the theoretical point of the next outside plate is reached. Where the flange plates vary in thickness, they shall decrease outward from the flange angles. Girders formed of web plates and angles alone, having no upper flange plate proper, will not be allowed. The total thickness of plates and angles shall not exceed five times the diameter of the rivet used. Flanges of plate girders over 12 ins. in width shall have at least four rows of rivets, and those over 16 ins. in width at least six rows of rivets.

All flange plates, subject to either tension or compression, spliced in the length of the girder, shall be covered with an extra amount of material equal in section to the material spliced, with sufficient rivets on either side to transmit the strains from the parts cut. Flange angles shall be spliced with angle covers.

In calculating the shearing or bearing strain in web rivets of plate girders, the whole of the shear acting on the side of the panel next to the abutment shall be considered as being transferred into the flange angles at a distance equal to the depth of the girders.

The webs of plate girders shall be spliced, wherever cut, by a plate on each side of the web capable of transmitting the full shearing strain through spliced rivets.

When the thickness of the web plates is less than $\frac{1}{60}$ of the unsupported distance between flange angles, heavy stiffeners shall be riveted on both sides of the web, with a close bearing against the upper and lower flanges and calculated as columns by the compression flange formula for the whole shear at the several points where they are placed. These stiffeners shall be placed at distances, center to center, generally not exceeding the depth of the full web plate, with a minimum limit of 4 ft. Web plates generally shall have stiffeners at bearing points and at points of concentrated loading.

Net sections shall be used in all cases in calculating tension members, and in deducting rivet holes they shall be taken as $\frac{1}{8}$ in. wider than nominal diameter of rivet. In calculating the net sections, having rivets staggered, all rows shall be deducted, unless so arranged that the net section along a zigzag line, taking all distances in the diagonal direction at only three-fourths their value, exceeds the corresponding net section directly across the plate.

Rivets shall not be spaced closer than three diameters, center to center, nor further apart, in the direction of the strain, than twelve times the thickness of the thinnest external plate connected, and not more than thirty times that thickness at right angles to the line of strain.

Rivets shall not be spaced closer to the side of the plates than $1\frac{1}{4}$ diameters to the center of the rivet, nor further from the side than eight times the thickness of plate. In no case shall the pitch of rivets exceed 6 ins.

Field rivets shall be reduced to a minimum.

Built chords shall be thoroughly spliced with rivets and additional section sufficient to transmit the entire strain; no allowance shall be made for abutting surfaces, except in heavy work.

When necessary to obtain sufficient bearing surface at pin-holes, reenforcing plates shall be added. These plates shall have a sufficient number of rivets to properly distribute the bearing strains from the pins to the member to which they are connected.

All segments of members in compression, connected by strapping only, shall have terminal cross-bracing plates at each end, the rivets and net section of which shall be sufficient to transfer the total maximum strain borne by the segment, and the thickness of

which shall not be less than $\frac{1}{40}$ of the distance between rivets connecting them to the compressed member. In no case, however, shall the length of the batten plate be less than the width of the member.

The distance between connections of strapping shall be such that the individual members composing the column, considered with hinged ends and a length equal to the distance between these connections, shall be stronger than the column as a whole, and in no case shall this distance exceed eight times the least width of these members. Where the ends of the compression member are forked to connect the pins, the strength of each leg shall be at least equal to the entire strength of the column, and the re-enforcing plates shall extend not less than 6 ins. beyond the edge of the batten plates.

Single lattice straps shall have a thickness of not less than $\frac{1}{40}$, and double lattice straps not less that $\frac{1}{50}$, of the distance between the rivets connecting them to the compressed member, and their widths shall be :

For	15-in.	channels or	$3\frac{1}{2}$-in.	and 4-in. angles ($\frac{7}{8}$-in. rivets).	$2\frac{3}{4}$ ins.
"	12-in.	"	" 3-in.	angles ($\frac{3}{4}$-in. rivets).....	$2\frac{1}{2}$ ins.
"	9-in.	"	" $2\frac{1}{2}$-in.	" ($\frac{3}{4}$-in. rivets).....	$2\frac{1}{4}$ ins.
"	8-in.	"	" 2-in.	" ($\frac{5}{8}$-in. and $\frac{3}{4}$-in. rivets).	2 ins.
"	7-in.	"	" 2-in.	" ($\frac{5}{8}$-in. and $\frac{3}{4}$-in. rivets).	2 ins.

Single lattice bars shall generally be inclined at an angle of 60° to the axis of the member, and double lattice bars at an angle of 45°, with a rivet at their intersection.

All bridges over 65 ft. long shall be provided at one end with turned friction rollers, not less than $2\frac{1}{2}$ ins. in diameter, between two planed surfaces. For spans of 65 ft. or less, planed surfaces shall be used without rollers. The nest of rollers shall be properly protected from the accumulation of dust and cinders.

Trusses shall be secured against side motion on bearing plates and rollers. The bolster blocks shall be joined to the truss, and the bearing plates shall be secured to the underlying supports by bolts or dowels.

Eye-bars shall be so packed as to produce the least bending moment on the pin, and shall not be packed out of line with the axis of the member more than $\frac{1}{8}$ in. to 1 ft.

No iron less than $\frac{3}{8}$ in. thick shall be used, except for packing or other idle material. No counter rod shall have less than $1\frac{1}{2}$ sq. ins. of sectional area.

The camber shall be such that under maximum load the bridge will not deflect below a horizontal position.

Quality of Material.

Rolled Steel.—Rolled steel shall be made by the open-hearth process, and shall contain not more than .04% phosphorus, .04% sulphur, nor .45% manganese. The steel shall be finished straight and smooth, and shall be a perfect product; the slightest flaw will be sufficient cause for rejection at any time during the progress of the work.

The tensile strength, yield point, and ductility of the material shall be determined from a standard test piece of not more than 2 ins. in width, nor less than ¼ sq. in. in sectional area, cut from a full-sized bar, and with sides turned or planed parallel, so as to give a uniform minimum section for a length of at least 12 ins.

Whenever practicable, the two sides of the test piece shall be left as they come from the rolls, but the finish on opposite sides shall be alike in this respect.

In determining the ductility, the elongation shall be measured after breaking, on an original length of 8 ins., in which length shall occur the curve of reduction each side of the point of fracture.

The yield point shall be that strain beyond which the elongation ceases to be proportional to the weight imposed, and may be indicated by "drop of beam." It shall in no case be less than 55% of the maximum strain sustained by the test piece. The speed of testing shall be governed by the inspector.

All rolled steel, except rivet steel, shall show by the standard test piece a maximum strain per square inch of 56 000 lbs. ± 4 000 lbs., with an elongation of 26% in 8 ins. Rivet steel, when tested in specimens of full size of rivet rod, shall show an ultimate strain per square inch not exceeding 56 000 lbs. with an elongation of 30% in 8 ins.

Each melt of finished material shall receive two tension tests—one cut from each extreme variation in thickness of metal rolled. When both tests comply with the specifications, all intermediate thicknesses will be accepted; otherwise only such thicknesses of metal will be accepted as show satisfactory tests.

Each finished piece of steel shall be marked with the melt number.

Wrought Iron.—All wrought iron shall be tough, ductile, fibrous and uniform in quality. It shall be thoroughly welded in rolling, and finished straight and smooth. It shall be free from flaws, blisters, cinder-spots, cracks, and imperfect edges. Scrap steel shall not be used in its manufacture.

The methods specified for testing rolled steel shall apply generally to wrought iron.

All iron shall show by the standard test piece a maximum strength of not less than 50 000 lbs. per square inch and an elongation of 20 per cent.

The yield point, as shown by the standard test piece, shall in no case be less than 26 000 lbs. per square inch.

All iron when cut into testing strips $1\frac{1}{2}$ ins. in width and with corners rounded to $\frac{1}{8}$ in. radius, must be capable of resisting, without signs of fracture, bending cold 90°, with the inner radius not to exceed three times the thickness of the test piece.

All iron which is to be bent in manufacture shall, in addition to the above requirements, be capable of bending sharply to a right angle at a working heat without any signs of fracture.

Cast Iron.—All cast iron shall be tough and sound, free from blow-holes, cold-shuts or other injurious imperfections. When broken the fracture shall indicate a good quality of gray iron.

Sample test specimens, 27 ins. long, 2 ins. × 1 in. in cross-section, cast under the same circumstances as those which attend the casting of the full-sized piece, shall sustain at the center, when resting flatwise upon two dull knife edges, spaced 24 ins. apart, a load of 2 000 lbs.; the load to be sustained two minutes, and show a deflection of not less than $\frac{1}{4}$ in. before fracture.

Workmanship.

All workmanship shall be first class. All parts exposed to view shall be neatly finished. All nuts shall be hexagonal.

In punching, the diameter of the punch shall not exceed by more than $\frac{1}{16}$ in. the diameter of the rivet to be used, and the diameter of the die shall be as small as may be required to punch a clean hole.

The holes shall be so carefully spaced and punched that, upon assembling, no variation from a truly opposite position of more than $\frac{1}{16}$ in. will occur. All holes shall be reamed to an exact match before the rivet is driven.

Rivets, when driven, shall completely fill the holes, and shall be machine driven whenever possible. They shall have full, concentric, hemispherical heads of a depth at circumference of shank of not less than one-half the diameter of rivet, and with full bearing on the plates; or they shall be countersunk when so required. Rivet heads shall not be flattened to less than half the diameter of the rivet.

Generally the use of bolts instead of rivets will not be permitted, but when used in special cases the holes shall be reamed parallel and the bolts turned to a driving fit.

Where the work is to be field riveted, the parts shall be assembled before leaving the shops, and the holes reamed to match. In the case of splices of upper chords, or other compression members, they shall be brought to forcible contact by the use of turnbuckles, and after reaming shall have match marks put on the pieces so that they may be brought to proper position in the field, before riveting.

Finished members shall be true and free from kinks, twists and open joints.

Rods and bars which are to receive a thread shall be properly upset before the thread is cut. Where threads are cut on steel, they shall be properly filleted.

All members requiring adjustment shall be provided with sleeve nuts and check nuts. Open turnbuckles will not be allowed. The ends connected shall be distinctly punch-marked, at a distance of 12 ins. from the screw ends, so that these ends may be accurately located inside the nut.

Heads of eye-bars shall be of sufficient section to break the bar in every case. Bars shall be full size at the neck; no patching will be allowed.

Welds in the body of the bars or rods will not be allowed.

Pins shall be turned, perfectly finished, and straight.

All members having bearing on pins shall be carefully bored at right angles to the axis, unless otherwise shown in the drawings. No variation will be allowed between diameter of pin and pin hole of more than $\frac{1}{50}$ in. For pin holes, in pieces which are not adjustable for lengths, no variation of more than $\frac{1}{64}$ in. in length between centers of pin holes will be allowed.

Eye-bars shall be perfectly straight before boring ; the holes shall be in the center of the heads and on the center line of the bar. Bars which belong to the same member shall be bored at the same temperature and in one operation. They shall be marked for erection, so that they may be used in the same member.

When bolts are used instead of pins, a variation of $\frac{1}{16}$ in. will be allowed between diameter of bolt and hole.

All abutting surfaces, except flanges of plate girders, shall be neatly planed or turned perpendicular to the direction of the strain, so as to insure even bearings.

Stiffeners of plate girders shall be milled to fit tightly against the flange angles, and shall be packed straight. The packing shall fit close to the flange angles, leaving no open space.

Rollers shall be turned and roller beds and bed plates planed; the bottom of shoes shall be planed exactly parallel to the center line of pin, unless otherwise shown in the drawings.

Thickening washers shall be used whenever required to pack the pin joints tight.

Pilot nuts shall be furnished for each size of pin to preserve the thread of the pin, and to facilitate erection.

Painting.

All iron shall be scraped free from scale, and receive one coat of pure kettle-boiled linseed oil before leaving the shops, and one coat of approved paint after erection. All surfaces which come in contact,

or are enclosed, shall receive one coat of approved paint before being assembled.

All turned or faced surfaces shall receive a coat of white lead and tallow before leaving the shops.

Inspection.

Free access and information shall be given by the contractor for a thorough inspection of material and workmanship.

The inspector will make detailed reports of his inspection to the engineer, and may notify the contractor of any defects in material and workmanship, but all acceptances made by him shall be considered as temporary, and his inspection shall in no way relieve the contractor of full responsibility for the character and accuracy of the work until its completion and final acceptance by the engineer.

The contractor shall furnish without extra charge such standard test pieces as may be necessary to determine the uniform quality of the material, and also the use of a reliable testing machine with necessary labor for testing.

Full-sized members may be tested to destruction by the engineer. They will be paid for at cost, less their scrap value, if they fulfill the requirements, but when the requirements are not complied with, the tests shall be at the expense of the contractor. Such rejections of the finished material as he may consider warranted by the results of these tests will be made by the engineer.

DISCUSSION.

Mr. Breithaupt. WILLIAM H. BREITHAUPT, M. Am. Soc. C. E.—During the years 1859 to 1870 Herr A. Wöhler, a mechanical engineer on railroads, made for the Prussian Government a series of minute and careful experiments on the effect of often-repeated straining of iron and steel, by which he established what is known as Wöhler's law, which is an expression of what is characteristically called the fatigue of metals. Wöhler's conclusions were confirmed by Professor Spangenberg, who continued the investigation after 1870. The facts ascertained were promptly made use of by Gerber, Launhardt, who established formulas since known by his name, and others.

The author, in his clear analysis of Wöhler's experiments, deduces a basis of proportioning, and leaves little ground for objection to the theory of the fatigue of metals. As to the basis of proportioning, there is still difference among engineers. In the opinion of the writer, fixing the live-load allowable unit stress at only half that of the dead load gives a result which is too low. Bauschinger demonstrated, as referred to by the author, that, within limits, material does recover from strain, and it must be assumed that this recovery is more or less according to the time allowed for it. Wöhler made applications of stress at the rate of 72 per minute. Even at this rate constantly maintained, it will be noted that it takes over a year to make 40 000 000 repetitions. Such conditions obtain in machines, but not in structures. It should be said here that of Wöhler's experiments a few isolated ones were on the effect of variation of time intervals between applications of stress. In two sets of bending tests, the specimens being held at one end only, the results were as follows:

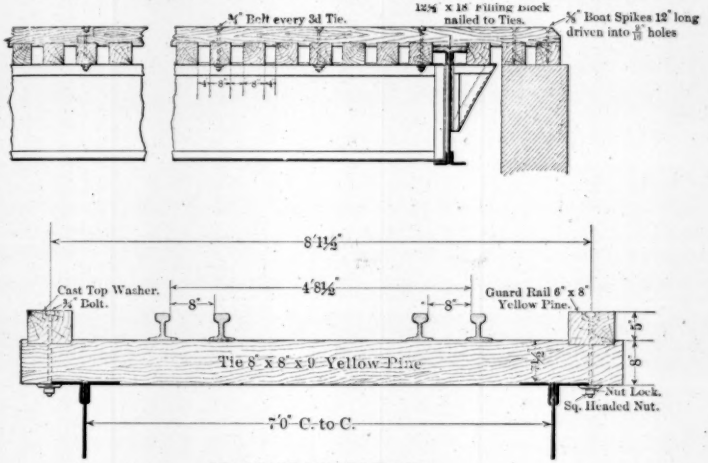
Maximum stress in pounds.	Number of Applications to Break Specimen.	
	18 per minute.	72 per minute.
29 960	170 900	183 145
25 680	610 000	909 810

Even with the longer intervals the applications are so frequent as to exclude any idea of recovery of material. The differences in the number of repetitions do not agree proportionally, and at best these few experiments are not conclusive. Wöhler himself modified his coefficients for cases of maximum loading applied only at long intervals. Launhardt's formula gives smaller sections than does the author's method, and so do the unit stresses adopted by other engineers, although in general not differing much.

Mr. Breithaupt.

The author's elimination of continuous drawbridges can apply economically, as no doubt it is meant to, only to turn-table draw-bridges of shorter span. Instead of the latter, single or double-leaf bascule bridges, as now developed, are in most cases preferable.

The author fixes the distance from center to center of track on double-track bridges at 12 ft. 6 ins. It should be whatever it is on the road. Ten years ago 12 ft. between centers on double track was common, now it has been increased to 13 ft. on a number of roads. There should be no change in alignment on or near a bridge if practicable to avoid it. Neither should there be change of grade. An abrupt change of grade is a fruitful cause of the parting of freight trains, of pulling-out of draw bars, of derailment, and of consequent



STANDARD BRIDGE FLOOR

FIG. 3.

injury or wrecking of structures. There is naturally a down grade toward watercourses, and the resulting changes of grade should be eased, and located as far away from the bridge as practicable.

If a single driver axle of the specified weight is carried by two 6 x 8-in. ties only, the extreme fiber stress per square inch with 7-ft. stringer spacing will exceed 1 000 lbs. by more than 400 lbs. With an 80-lb. or even a 75-lb. rail, it is usual, and is allowable, to consider the axle as carried by three ties. For a stiff floor, reasonably continuous in case of derailment, the ties should be spaced not more than 4 ins. apart in the clear. The guard-rails should have an even surface, and not be obstructed by washers or bolt heads. The desirability of this is obvious in considering derailment, or in the case of anything

Mr. Breithaupt.

dragging on the guard-rail which would be caught and thrown about by projections thereon. Over the floor beams (a space of 12 ins. or more), some support to the rail is desirable to avoid undue stress in the adjacent ties. This can readily be given by a filling block nailed in. A bridge floor as specified by the writer is shown in Fig. 3.

The live load given by the author is a typical one as used in present designing. The percentages for impact give increase for floor proportioning. Another method used* is to assume a uniformly heavy load over the entire bridge, with one local concentration only. Very heavy cars are to some extent coming into use, as referred to by the author, but in any event it must be assumed that engine loads will continue to be heavier than train loads, so that for a length of 100 ft. at the head of the train the load will be heavier than for the rest of the train. Proportioning on the basis specified by the author is therefore more rational.

On the subject of impact there is considerable agitation at present, and it is to be hoped that we may have more definite results. The author well says that there is as yet little or nothing on which to base definite specifications. It is, however, difficult to see why, if for 40-ft. spans impact is taken at 10%, it should be only 1% for 80-ft. spans. The writer has specified for impact as follows: 20% for stringers, 15% for floor beams, and for main girders in the proportion of 10% for 40-ft. spans to 5% for 100-ft. spans.

In riveting there are a number of things that affect the value of the rivet, such as slight mismatching or misfit of holes, short or over-length of rivet, under-heating, eccentric or otherwise careless driving, etc.; on the other hand, their value is largely added to by friction between the surfaces they hold together. In many cases the number of rivets is determined by their permissible minimum spacing. For field rivets, which are usually hand-driven, and the bulk of which are for floor connections, with a large excess of live-load stress, a proportional allowance on the side of safety is made. Nevertheless, the author's method of proportioning for rivets and details throughout in the same general way as for main members is the more rational one.

Unit dead-load stresses are given for shear and bearing on pins, rivets and bolts which are correspondingly somewhat higher than those allowed by most engineers. But there is no special value given for bending stress in pins. If the general dead-load unit stress is taken for this, as seems the intention, it will be inconsistent with the values given for shear and bearing, and furthermore, will give unnecessarily large pins.

The author is to be commended for not specifying any difference in unit stress between plates and shapes and eye-bars. It has

* Instance, Pennsylvania Lines West of Pittsburg Specifications.

been repeatedly shown* that shapes, when properly connected, are as good for net section as are eye-bars, if not a trifle better on account of the annealing of the latter. Mr. Breithaupt.

The pressure on rollers in pounds per lineal inch is specified as not to exceed $500\sqrt{d}$. With the minimum dimension of rollers specified, this does not apply for short single-track spans; for longer spans it is a small value. It has been found by experiment† that $1\,200\sqrt{d}$ lbs. is entirely safe, and this has appeared in bridge specifications. A common specification for this value is $300\ d$.

Provision for expansion ends does not generally receive proper attention. Frequent observation of dislocation of expansion rollers, in long and short spans indifferently, by the accumulation of dust or sand blown in between the rollers, has led the writer to adopt sliding expansion ends for spans up to 90 ft., and, for roller ends, ribbed bed-plates, made of small I-beams or of matched rails of a well-known pattern, having, under the roller surface, longitudinal recesses, open above; and to give all expansion bed-plates, whether for sliding or rolling, central guide ridges fitting into grooves on the rollers.

In conclusion, the writer does not understand why there should not be agreement and greater conciseness among engineers in the use of two certain words. To convey the same meaning, Stoney, an old and still valuable authority on many things, used the word "strain"; Rankine, later, used "stress"; Du Bois, of present American writers, uses or has used both "strain" and "stress"; Burr uses "stress," Johnson "stress," and so on. Most engineering writers, of text books, at least, if not of specifications, now agree in the use of "stress." Among engineers, generally, there is not such uniformity, and in engineering periodicals the confusion is apparent. Within the past two weeks one of the latter, in general notable for good matter and language, said "strain," and elsewhere in the same number "stress." for expression of identical meaning. Rankine, than whom no one is more entitled to fix engineering nomenclature, adopted the word "stress" for force applied to a body, and "strain" for alterations, of what nature soever, in the volume and figure of a solid body, produced by forces applied to it.‡ Under this definition, then, there is no such thing as strain per square inch as little as there is stress in length or width, or in the internal alterations of arrangement of the molecules of the body. There are many uses of "strain" to which, even in popular language, "stress" does not apply at all. Strain and straining are very convenient general terms for torsion, bending, tension or compression. The autographic record of such

* "Recent Tests of Bridge Members," by J. E. Greiner, *Transactions*, Am. Soc. C. E., Vol. xxxviii, 1897, p. 41.

† Crandall, Cornell.

‡ Rankine, "Applied Mechanics."

Mr. Breithaupt. an apparatus as the old Thurston testing machine, or of the extensometer—for which strainometer would be a better name, since it measures both extension and compression—is a strain diagram or strain sheet, while a diagram of a structure showing the forces applied to the various members is a stress sheet.

Mr. Merriman. MANSFIELD MERRIMAN, M. Am. Soc. C. E.—The speaker agrees with one or two of the written discussions in which it is claimed that the elastic limit is a more suitable point from which to reckon working stresses than the ultimate strength. The lowest ultimate strength allowed for steel by the author's specifications is 52 000 lbs. per square inch, and the lowest elastic limit about 28 600 lbs. per square inch. The latter figure indicates the degree of security afforded by the specified working stresses in a very much better manner than the former.

Wöhler's experiments determined smaller ultimate strengths under a large number of repeated loads, and Launhardt's formula is merely a convenient method of expressing these values after division by a factor of safety. Without a knowledge of the elastic limit, however, it is difficult to arrive at a sure decision whether Launhardt's values are or are not too high. Whether the stresses be steady or repeated, the elastic limit is to be regarded as a constant quantity, provided that none of the stresses surpasses that limit.

It is very much to be desired that an extended series of tests, like those of Wöhler, should be undertaken in accordance with modern laboratory methods. The author, in his discussion of Wöhler's results, appears to have found difficulty in ascertaining the tensile strength of the metal under a steady pull. This would not be the case in any future series of experiments, as specimens from the same bar would be tested under both steady and repeated loads. The tests carried on at the Watertown arsenal since 1888 by Mr. James Howard fulfill all requirements as to care and precision, but they cover only the case of rotating bars under transverse stress. They furnish no data for comparison with Launhardt's formula, since the fiber stresses alternate equal distances in tension and compression, but with respect to the question of fatigue they appear to harmonize with Wöhler's conclusions.

In one of these Watertown tests cast iron showed a remarkable power of resistance to fatigue. The specimens were gun iron, having an ultimate tensile strength of 31 200 lbs. and an apparent elastic limit of about 13 000 lbs. per square inch. Under alternating fiber stresses of 25 000 lbs. per square inch, applied at the rate of 400 repetitions per minute, rupture occurred after 4 700 repetitions. Under 20 000 lbs. per square inch rupture occurred after 26 400 repetitions. Under 15 000 lbs. per square inch rupture occurred after 47 283 500 repetitions. It is difficult to draw general conclusions from a single series of experiments like this, but it certainly shows that this cast

iron, under alternating stresses very near to its apparent elastic limit, had a resistance to fatigue almost, if not fully, equal to many of the steel specimens under similar conditions. Mr. Merriman.

The author's discussion of the question of fatigue seems unnecessary since the stresses in the members of a bridge truss should be kept below the elastic limit. and hence fatigue cannot result. In fact, the ultimate strength as determined by a steady pull appears to have a far greater value than the ultimate strength as found from an enormous number of repeated stresses, provided the actual working stresses are kept below the elastic limit. In respect to high and unusual stresses caused by unbalanced locomotive drivers moving at high speed, or by derailed trains, there may be a slight amount of occasional fatigue; but these stresses should properly not be taken into account in general rules applying to the normal and usual stresses.

The speaker does not wish to be understood as objecting to the author's rule which makes the effect of a computed live-load stress double that of a dead-load stress. If the live load be applied with perfect suddenness, this rule has a sound theoretical basis, and, although the factor 2 is theoretically too large for railroad trains, yet it is a good figure to use in order to provide for contingencies and for future increase in live loads. The author's general method of reducing all stresses to an equivalent dead-load stress, and of using the single value, 18 000 lbs. per square inch, as the allowable unit stress, seems a good and logical one.

The percentages allowed for impact, in the case of members whose maximum stress is attained by a movement of less than 80 ft. in the live load, are unfortunately arbitrary rather than experimental. Moreover, in counters it makes some difference whether these percentages are added to the computed stresses or to the live load, the former being implied in the first sentence of the third paragraph on page 156, and the latter in the statement that I is to be applied to the moving load.

The author uses the column formula in a very rational manner, the numerator P in the expression

$$p = \frac{P}{1 + q\frac{l^2}{r^2}}$$

being regarded as the true measure of the internal stress on the concave side of the deflected column. The speaker has urged this view for many years, having presented it in the first edition of "Mechanics of Materials" in 1885. The value of q properly depends upon the end conditions of the column; but, as the author's specifications make no distinctions in this respect, $\frac{1}{18\,000}$ seems a fair general value, particularly when it is considered that $\frac{l}{r}$ is never very large in chords and

Mr. Merriman. end posts. In a steel column with fixed ends the speaker regards $\frac{1}{24000}$ as a good value for q, while in a column with one end fixed and the other hinged $\frac{1}{14000}$ seems preferable.

It is unfortunate, perhaps, that in discussing a paper of this kind the temptation to criticism is stronger than the inclination to approve and commend. Hence, the speaker desires to say that he considers the specifications of the author clear, concise, carefully considered and well adapted to produce bridge structures of good workmanship and of ample security.

Mr. Schneider. C. C. SCHNEIDER, M. Am. Soc. C. E.—While reading that portion of this interesting paper, relating to the Launhardt formula, the speaker was reminded of a sentence in a letter by the late Professor Winkler, of Berlin. That letter is dated Jan. 19th, 1888, more than ten years ago, and in it he says:

"We have at present our trouble to abolish the errors which were caused by the abuse of Wöhler's experiments in determining permissible working strains, since Bauschinger has proved that repeated strains have a detrimental influence only above the elastic limit and alternate strains up to near the elastic limit."

It seems, however, that, in spite of Professor Winkler's efforts, these errors have not yet been eradicated. The experiments selected by the author do not appear to prove either the fatigue of metal inside the elastic limit, or his theory that a live-load strain produces twice the effect of a dead-load strain.

Wöhler's duration experiments were made for the purpose of determining the ultimate strength of the material, under repeated strains in one as well as in alternate directions. Based on these experiments, many elaborate theories were advanced by theorists and conveyed into more or less complex mathematical formulas, to be used for determining permissible working strains, the theorists entirely losing sight of the fact that their theories were based on results above the elastic limit to be used for conditions within the elastic limit, as they exist in actual practice.

The well-established theory of the elastic line is based on strains below the elastic limit, *i. e.*, the limit within which the elongation is proportional to the strain. As a single strain above the elastic limit produces a permanent set and destroys the property of uniform elongation in the metal, the effect of a single permanent strain is not different from the effects of repeated strains, as the single strain has practically destroyed the usefulness of the material and will ultimately produce rupture, if repeated often enough. The elastic limit, therefore, is actually the ultimate strength, for all practical purposes. Tests of material for rupture should be used only for the purpose of establishing its quality, and not for deducing laws or formulas for working strains for either tension or compression members.

The author has selected three of Wöhler's experiments on wrought

iron by which to prove his theories. The first one appears to prove that repeated alternate strains of equal intensity in both directions produce twice the effect of repeated strains in one direction. Mr. Schneider.

The second experiment proves that repeated strains of one kind, within the elastic limit, have no more detrimental effect upon the metal than a single strain.

The third experiment, on which the author relies particularly to prove that a live-load produces twice the effect of a dead-load strain, appears to be of no value for the purpose of deducing a law, as the maximum strain to which the metal was subjected was 47 080 lbs. per square inch (very nearly the ultimate strength). This would practically destroy the material on the first application, and the experiment was continued on material which had already lost its property of uniform elongation.

The author does not consistently follow his own theories, as he considers wind strains to be dead strains, which, according to his own definition, they are not.

Bauschinger's very carefully conducted and reliable experiments, with metal subjected to repeated and alternate strains, appear to prove that the difference between the elastic limit for alternate strains of equal intensity in both directions and repeated strains in one direction is far less than was generally accepted on the strength of the few experiments made by Wöhler.

In Table No. 1 the results of some of Bauschinger's experiments on the resistance of various kinds of metals to repeated and alternate strains are given. Therein may be found the ultimate strength or strains which produced rupture in one application, the strains which resisted an infinite number of repeated applications and the strains which resisted an infinite number of repeated alternate applications of the same intensity in both directions. The values given are in pounds per square inch, and are approximate only.*

TABLE No. 1.

	Ultimate strength.	Repeated strains.	Alternate strains.
1. Wrought iron	49 500	28 450	25 170
2. Ingot iron	62 000	34 140	28 160
3. Material not specified	57 600	31 290	28 160
4. " " "	57 180	34 130	32 140
5. Thomas steel	87 050	42 670	42 670
6. Rail steel	84 480	39 820	39 820
7. Boiler plate, ingot iron	57 600	34 130	27 000
8. Material not specified	47 650	31 290	22 750

* Föppel's Technische Mechanik.

Mr. Schneider. These experiments indicate that alternate strains reduce the elastic limit below that established by strains in one direction, and, if the working strains were kept within that lower elastic limit, it would not be necessary to consider the strains in the opposite direction, and, therefore, the absolute maximum strain could be used in determining the section of a member.

All the results of experiments appear to prove conclusively that within the elastic limit all strains have the same effect; the internal strain of a member being proportional to its elongation or reduction. It is therefore evident, as far as the resistance of the material is concerned within the elastic limit, that it makes no difference whether this strain is produced by the weight of the structure, the static effect of a superimposed load, or the dynamic effect of a moving load.

A structure is safe if it is able to resist the exterior forces acting upon it for an infinite period. To accomplish this all members of the structure should be so proportioned that the material composing them will be able to resist permanently the strains produced by the exterior forces.

Now the question arises : What are the exterior forces acting upon a railroad bridge ? They are :

1. The weight of the structure.
2. The static effect of the moving load.
3. The dynamic effect of the moving load, commonly called impact.
4. The strains produced by wind, centrifugal force and momentum of train.

As experiments have shown that strains in the members of a bridge produced by a fast-moving train are greater than those produced by the static load of the same train, it is necessary to recognize the fact that the strains are increased by the dynamic effect or impact of the moving engine and train. This fact has also been recognized by engineers and considered in specifications.

Some of the older specifications required that a percentage for impact be added to the live-load strains for stringers, floor beams and hangers. The speaker believes that Joseph M. Wilson, M. Am. Soc. C. E., was the first one in the United States who considered the impact for all spans, in his specifications, published in 1885.* These made it dependent upon the proportion of live to dead load, for which purpose a formula is given, which, though similar to the Launhardt formula in form, is, however, different in the results.

Many other engineers have followed Mr. Wilson's example.

Theodore Cooper, M. Am. Soc. C. E., in his specifications for railroad bridges allows for live load only half the permissible working strain he does for dead load, which is practically the same as using a uniform working strain for all loads and adding 100% for impact to all live-load strains, irrespective of the length of the span.

* *Transactions*, Am. Soc. C. E., Vol. xv, p. 390.

Mr. Schneider.

The speaker in his practice uses a uniform working strain, and increases the live-load strains by adding a percentage for impact to the strains produced by the static effect of the live load, making the effect of impact dependent on the length of the span.

The author also considers the dynamic effect of the moving load, and makes provision for impact in spans up to 80 ft.

If impact is considered an established fact, as the author evidently does, then it must be admitted that it exists in all spans.

As the experiments made, up to the present time, do not give sufficient data for computing the dynamic effects of the live load with accuracy, it is necessary to use some practical judgment in this respect and to allow ample margin so as to be certainly on the side of safety.

If reference is made to the speaker's specifications, and the percentages of live load assumed for impact are compared with the experiments made by Professor Fränkel, S. W. Robinson, M. Am. Soc. C. E.; J. E. Greiner, M. Am. Soc. C. E.; F. E. Turneaure, Assoc. Am. Soc. C. E., and others, it will be seen that such percentages are far in excess of the values found by the experiments. The speaker has already given his views on this subject in the discussion of Mr. Greiner's paper entitled, "What Is the Life of an Iron Railroad Bridge?"* and his reasons for adopting the impact method.

As most specifications in use at the present time make provisions for increased strains due to the dynamic effect of the live load in some way or other, either by disguising it in the shape of a formula, or by allowing different working strains for dead and live load, the speaker believes it would be more rational and logical to consider the dynamic effect of the live load separately, and, if in the future more reliable data for computing the impact should be obtained the numerical values of the coefficients could be changed, which would, however, in no way affect the principle of the system.

The speaker fully agrees with the author, that a uniform working strain should be used for all stresses, that strains allowed for tension should be the same as those for compression, and shearing and bearing strains in proportion; or, in other words, one square inch of section should represent a certain number of pounds of strain. This seems to be the most rational method which will insure the proper proportions of details and connections in relation to the main members of a structure. The speaker has used this method in his practice for the last twelve years and has found it to work very satisfactorily.

The author specifies a working strain of 18 000 lbs. per square inch; the speaker uses in his practice a working strain of half the

* *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 328.

Mr. Schneider.

elastic limit, or a factor of safety of two. This would give, for the steel specified by the author, with an average elastic limit of about 30 000 lbs. per square inch, a working strain of 15 000 lbs. per square inch.

The method proposed in the author's specifications for proportioning the members of railroad bridges, may appear entirely different from the speaker's practice, the author multiplying the live load by two, adding an additional variable percentage of impact for spans up to 80 ft,, and using a working strain of 18 000 lbs. per square inch, while the speaker adds a variable percentage for impact to the live load for all spans, using a working strain of 15 000 lbs. per square inch. However, if the results are compared, only small differences will be found in the main members of the bridges. The greatest differences occur in the lateral systems. The assumed wind strains being the same in both specifications, the difference consists in the specified working strains. The author allows 18 000 lbs. per square inch, while the speaker allows only 15 000 lbs. per square inch, making his lateral bracing 20% stronger than if computed in accordance with the author's specifications.

Table No. 2, computed by the speaker, gives the tensile strains per square inch, in the chords for different lengths of spans, produced by the total load, calculated in accordance with both specifications.

TABLE No. 2.

Length of span in feet.	Strain per Square Inch in Pounds.	
	Seaman.	Schneider.
10	7 170	7 850
50	9 120	8 660
100	10 380	9 680
200	10 870	10 760
300	11 270	11 540
400	11 640	12 150
500	12 100	12 680

On page 152 the author makes the following statement, to which the speaker takes exception:

"The author is not aware of any specification which has made this distinction between live and dead strains in proportioning details, except those which adopt the Launhardt formula throughout."

There were specifications in existence for many years which covered that point. In 1887 specifications were published by the Pencoyd Iron Works which made this distinction between live and dead-load strains in proportioning details without the use of the Launhardt formula.

Mr. Schneider.

Referring to the specifications proper, the speaker believes that if bridge work were done in accordance with these specifications first-class material would be obtained, but only average designs and workmanship. While the speaker agrees with the author in a general way, and is pleased to notice that the specifications are short and precise without any useless requirements which would only increase the cost of the work without improving its quality, they do not, however, come up to the speaker's ideas of the best modern practice as regards design and workmanship.

These specifications will not compel the manufacturer to make first-class designs, unless he is led to do so on his own account, because he takes pride in his work and has ability, experience and sufficient judgment, which the speaker regrets to be compelled to admit is not always the case. The same remark refers also to workmanship.

Under "General Provisions," the author says: "A type of truss shall be used in which the strains may be readily calculated, and which subjects no member to alternate strains."

This clause would prohibit a class of structures which in late years has given much satisfaction to engineers who have had charge of the maintenance of bridges, viz., the riveted-truss and lattice bridges. There are some riveted-lattice bridges in existence which were built over twenty-five years ago in which the workmanship is very poor, and yet they are doing good service at the present day; while pin-connected spans, built at the same time under similar conditions and specifications, have been replaced by new structures years ago. If experience counts for anything, the riveted-lattice bridge has certainly held its own; in fact, modern practice requires riveted bridges for spans greater than those for which plate girders are considered practicable, and up to those where pin-connected spans are advisable, generally between 100 and 130 ft. The speaker, therefore, is of the opinion that the portion of this clause referring to alternate strains should be omitted.

The author specifies: "Stringers and deck-plate girders shall not be spaced further apart, between centers, than their depth, with a minimum limit of 5 ft." There appears to be no good reason for specifying that limit, excepting that it was formerly the practice on the Pennsylvania Railroad. It was abandoned years ago. For double-track bridges the usual practice is to space the stringers 6½ ft. apart between centers (the distance between centers of tracks being 13 ft.), which, in the speaker's opinion, should be the minimum distance between centers of stringers or deck-plate girders.

Under "Loads" the author specifies that provision shall be made for the sudden starting or stopping of a train, estimating the coefficient of sliding friction at 15 per cent. The almost universal

Mr. Schneider. practice is to use a coefficient of friction of 20 per cent. As in some cases the train may be brought to a sudden stop by reversing the engine and applying sand to the rails in addition to using the brakes, in which case the coefficient of friction would be over 30%, the writer would recommend adherence to the usual practice.

In the formula for compression members the author uses a coefficient of 18 000 in the denominator. The speaker would suggest 10 000 instead, so as not to encourage long, slender compression members, and for the same reason would recommend the following additional clause:

"No compression member shall have a length exceeding 100 times its least radius of gyration, excepting those for wind bracing, which may have a length not exceeding 120 times the least radius of gyration."

The speaker would also recommend that the least width of posts in pin-connected bridges be limited to 12 ins., in order to offer some resistance in case of a derailment. The formula for members subject to alternate strains is somewhat confusing, as in this formula p denotes the direct compression in the member, while in the formula for members subject to compression only, p denotes the allowable strain per square inch. Instead of using a complex formula, practically the same results would be obtained by substituting the following clause:

"Members subject to alternate strains of tension and compression shall be so proportioned that the total sectional area is equal to the sum of the areas required for each strain."

The author limits the working strain on rivets in tension to one-half the limit allowed for direct shear. As the author specifies steel for rivets, and as recent experiments have shown that steel rivets in tension give just the same results as round bars of the same diameter, invariably breaking in the shank, there appears to be no good reason for such a low limiting strain. The speaker would suggest that the allowable tensile strain on rivets be made the same as that for direct shear.

The author has omitted to specify the working strain allowed for bending on pins, which, in accordance with good practice, may be the same as that for bearing.

The author specifies that expansion rollers and bed plates should be proportioned for the total load. If the effect of the live load is different from that of the dead load, why should this not apply also to rollers and bed plates?

Under "Details of Design" the author specifies that: "Each main panel of deck-bridges shall be provided with intermediate sway-bracing rods." Modern practice requires stiff bracing.

Referring to trestle towers, the author says: "The struts shall be proportioned to withstand their component of strain." This evidently refers to trestle towers with adjustable rods. The speaker is not aware of any first-class railroad which would accept trestle towers with

adjustable rods at the present time. This should, therefore, be considered antiquated construction and not in accordance with the best modern practice. Mr. Schneider.

The author specifies: "Girders formed of web plates and angles alone, having no upper flange plate proper, will not be allowed."

There appears to be no good reason why girders without an upper flange plate should not be allowed, as all theoretical and practical considerations point the other way, excepting that it was once the practice on the Pennsylvania Railroad. This restriction has been abandoned by this railroad as well as by most other railroads which followed that practice, as being a nuisance to the Maintenance of Way Department. The general tendency of modern practice seems to be to allow no upper flange plates on girders which carry ties on the upper flange, as far as this is practicable.

The speaker thinks the following clause should be omitted: "Flanges of plate girders over 12 ins. in width shall have at least four rows of rivets, and those over 16 ins. in width at least six rows of rivets," as in his experience he has had occasion to observe that this very clause, being put in the specifications as a cast-iron rule without qualification, has produced in some instances very ridiculous results.

In accordance with the author's rule for proportioning plate girders, a girder 24 ins. deep with a web plate $\frac{5}{8}$ in. thick would require stiffeners every 2 ft. In the speaker's opinion the intermediate stiffeners in a girder of such proportions might be dispensed with.

The author's rule for determining the length of tie-plates at the ends of compression members, whose segments are connected by latticing, seems to be made to accommodate the poorest workmanship, as even moderately good workmanship (as long as the pin is bored at right angles to the center line of the member) will distribute the pressure over both segments. Strict conformity to this rule, which is also found in some other specifications, has produced in some cases very anomalous results. The speaker is of the opinion that if the end tie-plates are made as long as the width of the member, it will be sufficient for all cases.

The author specifies that: "All bridges over 65 ft. long shall be provided at one end with turned friction rollers, not less than $2\frac{1}{2}$ ins. in diameter."

The speaker would suggest that for plate girders of ordinary length, that is, up to 80 ft., the expansion end should slide on planed surfaces; and, if friction rollers are used for plate girders, the ends should be supported on pin shoes in order to distribute the pressure over the rollers so as to make them effective. He would also limit the size of rollers to 3 ins. in diameter.

While the speaker is of the opinion that specifications should not restrict the designer in using his judgment in the selection of details,

Mr. Schneider. he believes, however, that they should confine him to good practice and substantial designs, but prohibit gimcrack construction. The speaker refers more particularly to those antiquated details, which were in vogue when bridge building in the United States was in its infancy, such as loop rods with bent eyes, wing plates, U-nuts, stirrups, loose hangers and other unsubstantial and complicated contrivances. The speaker is constrained to admit that this kind of flimsy construction is practiced by a few engineers, even at the present day, thus showing the necessity for prohibitory clauses in specifications. The few who still cling to the old practice are either those who copy their designs from books, or those who have for years been making routine designs for a bridge shop, and have therefore become so accustomed to work in the same rut in which they commenced years ago, that they are loath to depart from their old ways.

The following clauses inserted in the specifications would prevent this kind of construction:

"Adjustable members in any parts of structures shall preferably be avoided.

"All lateral and sway bracing shall be made of shapes which can resist tension as well as compression.

"All floor beams in through-bridges shall be riveted between the posts, above or below the pin."

The ideal modern structure has few or no adjustable members, and when once erected needs little attention excepting such as is necessary to protect the metal from corrosion.

Another practice which is not consistent with good design, and which is still persistently adhered to by a few designers, is the lop-sided construction, which the author's specifications do not seem to exclude. By lop-sided construction the speaker refers to members of pin-connected trusses with unsymmetrical sections, viz., sections in which the neutral axis is not in the center line of the member. Placing the pin in the neutral axis of an unsymmetrical section by no means distributes the strain uniformly over the entire section, but produces bending strains in addition to the direct compression. It is also unsightly and unconstructive. In order to prohibit this construction, the speaker would suggest the following clause:

"All main members in pin-connected trusses should have symmetrical sections, and the pins should be placed in the line of the neutral axis."

Most modern specifications prohibit the use of cast iron, and where it is necessary to use castings, cast steel has taken its place. The specifications for the quality of material should therefore include cast steel, which is not mentioned by the author. Under "Workmanship," the author specifies: "All workmanship shall be first class," but omits some particulars which are essential to first-class workmanship. The specifications say: "Where the work is to be field-riveted,

the parts shall be assembled before leaving the shops, and the holes reamed to match." This is practicable in only a few cases. The majority of field holes are in the connections of the floor system. The assembling and reaming of these holes in the shop is not only impracticable, but it is bad practice. As all members which are alike should be interchangeable, the field connection holes between floor beams and stringers and between floor beams and posts should be drilled or reamed to iron templates. Mr. Schneider.

The author specifies that: "All abutting surfaces, except flanges of plate girders, shall be neatly planed or turned," etc. Why except flanges of plate girders? The speaker is of the opinion that abutting surfaces in flanges of plate girders should be planed in order to make a good fit and give the work a neat and finished appearance, more particularly as this is the usual practice in first-class bridge shops. First-class workmanship requires also the ends of floor beams and stringers to be faced.

The author limits the play between pin and pin hole to $\frac{1}{50}$ in., but when bolts are used instead of pins a variation of $\frac{1}{16}$ in. is allowed. Why this distinction? If a bolt is used in place of a pin, it should have the same fit; and, if it is used in place of a rivet, it should be turned to a driving fit.

The speaker would suggest the following additional clauses under the heading of workmanship:

"All holes for field rivets, excepting those in connections of lateral and sway bracing, shall be accurately drilled to an iron template, or reamed while the connecting parts are temporarily put together.

"The ends of floor girders shall be faced true and square.

"All eye-bars or pieces which have been partly heated shall be annealed."

GEORGE S. MORISON, Past-President, Am. Soc. C. E.—The speaker believes the general practice which has been recommended in this paper, that the live load shall be computed as having twice the effect of the dead load, is one which has been generally recognized as good, for a considerable time. It is the one which he has followed. But whether it should be applied to all classes of strain seems exceedingly doubtful. All the formulas and all the experiments which lead to any such conclusions are based on the rupture of the metal; that is, they are based on the final destruction of the metal by tension. Now, supposing a member never receives anything but compression, it certainly, under that condition, is never going to break by tension. And yet every compression member that anyone has ever seen tested to rupture has broken by tension. This means that when a long compression member breaks, it is enduring strains which are very different from the strains to which it ought to be subjected when in actual work. If the strains in a compression member are kept down to such a limit that nothing but compression exists in that member, under the Mr. Morison.

Mr. Morison. working strains, there may be no reason for making any difference in the limit of strain allowed for dead and for live loads. There are two ways in which tension appears in a compression member. One is by the member being so long that, through various irregularities, it is deflected out of shape. The other is by the application of the strain outside of the axis of that member. Both of these conditions should be avoided in a well-designed bridge. It is possible to determine, by a careful examination with micrometers, whether the members of a bridge are straight; nothing but compression can exist in compression members so long as the members are found to be straight. In a well-designed structure, if the length of the posts is limited to thirty times the least transverse dimension, the chance of getting tension in the posts, under any working strains, is so small that there is no reason for making any difference in the strain permitted for dead and for live load. It has been the speaker's practice for a good many years to take particular pains to make the compression members balanced sections. In the vertical posts of an articulated pin-connected truss this can always be done. In posts there is danger of tension arising under various disturbing causes, especially as posts are liable, in case of derailment, or otherwise, to side blows. In the chords of an articulated truss it is also possible to balance the section so as to have practically the same resistance above and below the center line. The chords of a bridge are not liable to blows, and the length of chord sections can be kept down to something like fifteen times the least transverse dimension. These are points on which there is a great deal to be said, but it has been the speaker's practice for years, though making exactly the relative allowance that the writer does for the difference between dead and live load in tension, to allow a uniform strain in compression; but while doing so he makes rigid rules, on the lines which have just been referred to, in limiting the length of compression members.

The next matter is the load specified. This paper follows the usual practice of providing two locomotives followed by a certain train-load. Curious results have sometimes occurred in specifications drawn on this basis. A specification was sent out by a prominent western road some years ago in which a locomotive was carefully given, with length between wheels, and weights, followed by a certain train-load, and the train-load was several hundred pounds per lineal foot more than the locomotive averaged. The fact must be faced that there are likely to be very great changes in motive power at any time. There have been such changes. There will continue to be such changes. It is not at all improbable that in many localities trains will be operated by electric power, and trains will be practically of uniform weight throughout, with concentrated weights on wheel bases at various points in the train. In this connection the speaker will present, from a paper pre-

pared for his own private use, the conclusions which he drew some years ago as to the proper basis for live loads; this being intended not as a basis adapted to any rolling stock actually in existence, but to the probabilities which must be faced, these conclusions being: Mr. Morison.

"*First.*—It is inexpedient to proportion structures for any specific form of locomotive, as all such forms are liable to rapid change.

"*Second.*—The present indications are that the train-load per foot is likely to be as heavy as the locomotive load per foot, though on a maximum train-load the distribution is uniform; whereas, it is not so with a locomotive.

"*Third.*—The load on the driving wheel base of a locomotive is liable to be twice as much per lineal foot as the train-load or the average weight of the locomotive per lineal foot.

"*Fourth.*—The driving wheel base of a locomotive may be taken as equivalent to 20 ft., a uniform load extending over 20 ft. being equivalent to an equal aggregate load concentrated on three driving wheel axles 6 ft. 8 ins., between centers, or on four driving wheel axles 5 ft. between centers, or on five driving wheel axles 4 ft. between centers.

"*Fifth.*—On fast passenger locomotives extreme weights are liable to be concentrated on a single pair of driving wheels, these weights being increased by an injudicious system of counter-balancing; to meet this condition one-half the entire driving wheel load should be considered as concentrated on a single axle.

"*Sixth.*—The usual length of the heaviest class of locomotives, as coupled up in the train, is about 60 ft.

"*Seventh.*—The loads may be considered as practically uniform for all lengths exceeding the length of two locomotives (120 ft.); for lesser lengths there should be a provision made for an increased load corresponding to irregular loads and such local disturbances as are caused by flat wheels and other imperfections.

"*Eighth.*—Provision should be made for the rapidity with which strains are put on the web members, this provision increasing from the ends toward the center on a basis which provides an excessive load for those portions of the strains which are greater when the bridge is partially loaded than when fully loaded.

"*Ninth.*—It is expedient to adopt different classes of weights or loads for structures which carry different classes of traffic, but in the interest of simplicity and uniformity it is advisable to observe the same rates of variation between train-load, wheel-base load, etc."

The speaker has followed this practice for years, and it has given very satisfactory results, and wherever he has had occasion to compare it with some special loading which has come out, he has been surprised to see how closely it has agreedwith that load. A prominent instance was the case of the special trucks on which the Krupp gun was taken to Chicago during the World's Fair.

The next point to which he would refer is the design of plate girders; and he thinks that the specifications call for a class of work which is not the best. The nearer a plate girder can be made to an absolutely solid piece of metal of its dimensions, the more perfect it is. The fewer pieces a plate girder can be made of, the more nearly it will approach this condition. A plate girder made simply of a web plate and four angles is to be preferred to one with cover plates above

Mr. Morison. and below. There are advantages in cover plates, but every particle of strain that goes into a cover plate must be transferred from the web through two sets of rivets instead of one. In the case of plate girders, like some of the older ones and like some that unfortunately are still made, in which a large number of cover plates are piled up on the two flanges, top and bottom, the strains in the outer of which plates must be transferred by rivets of a length several times their diameter, there are serious doubts whether these outer plates really do much work. According to the theory on which plate girders are proportioned, the farther the metal is from the neutral axis, the greater the amount of work it does. But when the work can only be performed through strains transmitted by long, slender rivets, which rivets can bend, no matter how perfect the work is, the speaker believes that the amount of work done by the cover plates, instead of increasing with the distance from the neutral axis, decreases with that distance. Everything which encourages the use of thick webs in plate girders and a small number of parts in the flanges is to be approved. The speaker, therefore, always proportions plate girders by the moment of inertia, taking into consideration the whole of the web to resist the bending strains; of course, this means the whole of the web as it contributes toward the moment of inertia. Eliminating the web encourages the use of thin webs, and there are many objections to thin webs. A thin web gets out of shape. It deteriorates more rapidly if it rusts. It is much more easily injured in case of any kind of accident.

Another feature in the specification for plate girders which seems to require some change is the matter of stiffeners. The speaker does not believe that it is right to consider a plate girder as an articulated structure. The web must be treated in the same way as the web of an I-beam. It is a continuous web, and the only function of the stiffeners is to act as stiffeners to prevent the web from buckling. There is no established good rule for determining the position of stiffeners; but the deeper the girder is, the closer the stiffeners should be together, if they are to act solely as stiffeners. The following rule is suggested, which is slightly varied from the one the speaker has usually followed: Where the depth of the girder exceeds 50 times the thickness of the web-plate, vertical stiffeners shall be used. The stiffeners shall be spaced at such distances that the product of the distance between the stiffeners by the depth of the girder shall not be more than 10 000 times the thickness of the web. That is, if the depth of the girder was 100 times the thickness of the plate, the difference between the stiffeners would be the same as the depth of the girder.

There is one other point. This specification contains the old provision that the rollers shall be proportioned according to the square root of their diameters. This is entirely wrong. Furthermore, any provision like the one contained in these specifications that the rollers

shall be so enclosed as to exclude dust is simply shutting in what you do not see. Some years ago the speaker took great pains with a roller bearing; it had a planed steel plate at top and bottom, and the ends of the rollers were enclosed by rubber gaskets under brass plates. It looked all right. When it was examined the rubber gaskets had served admirably to keep the dust in. There is no way of excluding dust which does any good. The only satisfactory method is to make a construction in which the dust will blow out a little more easily than it will blow in; and this can be accomplished by placing the rollers on an open bed, made, as the speaker prefers, of rails placed about $\frac{1}{2}$ in. apart and planed, in which there is free chance for the wind to blow in all directions, and in which, when the dust blows through the cracks, it falls into a space a good deal bigger than the crack, from which the wind will clear it out entirely. The speaker has never had any trouble with dust collecting around rollers since he has used this detail. Mr. Morison.

But to speak of the pressure on rollers, he thinks that the correct conclusion is briefly stated in these words, which were written five years ago:

"The American practice has generally been to make the permissible weight per lineal inch of the roller proportional to the square root of the diameter of the roller. A common provision in specifications limits the weight in pounds to $500\sqrt{d}$, d being the diameter of the roller. This rule is based on the theory that there is a permissible limit of indentation, which the roller may be allowed to make in the rolling surface, which permissible indentation is supposed not to exceed the elastic limit. If the versed sine of the indentation is constant, the chord of the same indentation will vary as the square root of the radius of the circle; and as the permissible weight is supposed to be a constant of the chord of indentation, which chord varies as the square root of the radius or diameter, the strain is determined accordingly.

"While mathematically correct, this rule seems to me based on an incorrect conception of the function of rollers. There is no special reason why the strains, either on the surface of the rollers or on the bearing surface, should be kept within the elastic limit; on the other hand, it would be better to have them exceed the elastic limit, if the result of such excess was to reduce the surfaces, both of the rollers and the bearing plates, to the condition of the surface of cold-rolled iron. The only necessary limit is that the elastic limit must not be so greatly exceeded that the metal under the surface will flow, and the roller become permanently deformed, as has occurred in conical roller bearings under turn-table centers.

"On the other hand, the same absolute indentation which would lock a small roller so that it would lie stationary in a groove might do little harm to a large roller. If we assume the roller to be set in a groove, we have forces applied on a bent lever tending to lift that roller out of the groove, the vertical arm of the bent lever being the radius of the roller, and the horizontal arm half the width of the groove. So long as the width and depth of the groove are kept proportionate to the radius or diameter of the roller, the force required

Mr. Morison. to roll the roller out of the groove will be constant. In other words, the formula which will maintain for all sizes of rollers a constant freedom of motion would make the strains on those rollers proportional to their diameters.

"In Germany this rule has been followed, and while there has been a considerable variation in the unit strain allowed, the fact has been accepted that the weight placed on a roller should vary directly as the diameter of the roller.

"The American rule is not only incorrect in principle, but vicious in its results. It encourages small instead of large rollers, a 12-in. roller, for example, being allowed to carry only twice the weight that would be put on a 3-in. roller, so that the smaller the diameter of the rollers, the greater the weight which would be permitted on the same area of bearing surface.

"In proportioning expansion rollers, I have adopted a somewhat arbitrary basis, accepting $500\sqrt{d}$ as correct for a 4-in. roller, thus making the weight permissible per lineal inch 1 000 lbs. for a roller 4 ins. in diameter. Instead of varying the weight with the square root, I vary it with the diameter; this makes the permissible weight 250 lbs. per lineal inch on a 1-in. roller, and 3 000 lbs. per lineal inch on a 12-in. roller."

CORRESPONDENCE.

J. B. JOHNSON, M. Am. Soc. C. E.—The writer quite agrees with the author in interpreting all fatigue experiments as indicating that live loads require twice as great a factor of safety as dead loads. Indeed, in his "Materials of Construction" (1897), the writer undertook to show (Chap. XXVII), that the formula, Mr. Johnson.

$$p = \frac{a}{1 - \frac{\text{Min. stress.}}{2\,\text{Max. stress,}}}$$

was a true interpretation of Wöhler's results, where p = working maximum stress for combined loads, and a = working stress for live loads.

It is there shown, also, that this formula applies equally for stresses of the same and of opposite signs, and, furthermore, that it is neither more nor less than the old rule of twice as great a factor of safety for live as for dead loads. As this last is the burden of the paper, the writer can but agree with the author. The above formula replaces both the Launhardt and the Weyrauch formulas, as well as those which have long been in use by the Pennsylvania Railroad Company and others. The argument in its favor and the explanation of its derivation are given in full in the work cited, and they will not be repeated here, the significant thing being that the so-called "new method of dimensioning" is, when properly understood, simply the old rule long used by engineers, and given by Rankine, to use twice as great a working stress for dead loads as for live loads. When this is understood clearly a formula is hardly necessary, and the whole theory of a proper interpretation of fatigue experiments loses its iterest.

In the chapter cited a new theory as to the real character of the so-called "fatigue" is also advanced. It is the opinion of the writer that this should really be called the "gradual fracture of metals," instead of "fatigue of metals," since the metal away from the plane of rupture never seems to have been changed or injured in any way. His theory is that some one or more of the many small faults in the metal, sometimes called "micro-flaws," are the starting points for cracks, which gradually extend under the repeated elastic deformations until complete rupture occurs. If this is the case, it should not be expected that any great uniformity of results would be obtained on different specimens cut from the same bar, and this is found to be the case.

It is true that proper working stresses would never start the extension of these micro-flaws for any number of repetitions, but if the danger line can be fairly established, then engineers should keep well within it by the use of a suitable factor of safety; and certainly this danger line is very much lower for repeated or for reversed loads than for static loads. In fact, under static loads, these flaws would proba-

Mr. Johnson. bly never gradually extend under any load, however great. The difference between live and dead loads is, therefore, a vital one, however the theory of failure is regarded, and this distinction should certainly be introduced into the proportioning of all metal structures in some rational manner. Apparently no better rule can be found than the simple one of allowing twice as great working stresses for dead as for live loads.

Mr. Schaub. J. W. SCHAUB, M. Am. Soc. C. E.—Aside from the Launhardt formula, which the writer considers more clever than useful, there is no question but that it is rational to assume a different factor of safety for live load than for dead load, in proportioning the sectional areas required in bridge members. This matter was discussed* by the writer, in 1887, when he suggested in lieu of the Launhardt formula that a factor of safety of 3 be used for dead load and a factor of safety of 6 for live load. This suggestion has been very generally adopted by bridge engineers, and notably by Theodore Cooper, M. Am. Soc. C. E., in his "Specifications for Railway and Highway Bridges." If the author will refer to the Michigan Central Railroad specifications for 1896 he will find all his suggestions fully anticipated, including even the variable allowances for impact, excepting that no percentage of increase is added to the live load for spans exceeding 100 ft.

The use of different factors of safety for dead and live loads was first presented to the writer by Mr. Frank D. Moore about 1882, when in the employ of the late C. Shaler Smith, M. Am. Soc. C. E. At that time Mr. Moore used different factors of safety for dead and live-load stresses in chords of swing bridges, near the center, where the dead-load stress is so largely predominant when the case of the span swinging, without end supports, is combined with the live-load stresses.

Referring to the specifications proposed by the author, it might be suggested that in place of the concentrated loads a uniform load be used, combined with a single concentration so placed as to give maximum stresses. As to what this load shall be the writer is not prepared to say, inasmuch as there are a great many views on the subject.

Mr. Wright. CHARLES H. WRIGHT, M. Am. Soc. C. E.—The engineers employed in the offices of the several bridge companies would welcome more cordially than any other class the advent of a specification which would be at once acceptable to the engineers of the railroad companies, and to the bridge builders. These men also realize more keenly than others, perhaps, the apparent hopelessness of such a result being reached at this time.

The editor of a leading engineering journal once said: "No class of men are so intolerant of the ideas of their fellows as the bridge en-

* *Transactions*, Am. Soc. C. E., vol. xvii, p. 169.

gineers." This is in a measure true. The statement might be applied, however, with equal or greater force to other branches of engineering. The more progressive of the ship-builders frankly admit that the bridge-makers are far in advance of them in the design and manufacture of steel structures. If this be true, it must also be true that the bridge engineers, while insisting upon infusing something of their own individuality into their work, have been willing to accept, to some extent at least, the best from the work of others, and to engraft it in their own designs. As the men in the offices of the bridge companies recall (as they must all be able to do) how a set of designs will be heartily approved by one engineer, and be severed limb from limb by the next to whom they are submitted, they must still feel, however, that the time is not yet ripe for a universal specification. Notwithstanding the progress made in bridge engineering, there is still a long transition period ahead of it. Mr. Wright.

Only a few months ago the bridge companies were anxiously considering how best to meet the requirement of drilled medium steel. Expensive machines for doing this work will soon be idle, if, as seems likely, the present tendency toward punched, unreamed, soft steels continues.

The most a specification like that proposed can accomplish, at present is to eliminate the meaningless requirements, and more or less absurd clauses which punctuate the majority of the great number of specifications now in use.

Why should an arbitrary assumption of a factor of safety of 2, 3 or 4 be made, or imaginary typical (?) engine concentrations be used, and the stresses then be figured with such minuteness that, if the water bucket happens to be at the wrong end of the engine tender, all the computations are false?

Why should it be necessary in determining pin moments to use a specification which makes it necessary to find new allowed unit stresses at each point where a bar, or other member, rests on the pin?

Why should it be required that hours be spent in figuring dead-load deflections, and changes due to camber, for the sole purpose of making elaborate diagrams showing the notching (even to eighths and sixteenths of an inch) required in framing the ties, only to have it develop later that the bridge carpenter has found it necessary to take out a hump, or sag, of $\frac{1}{2}$ in., or more, sighting with his eye only?

There is no part of bridge anatomy which has not been made the hobby of this, or that, engineer. With one the trough, or hat shape, is the only type of chord to use in a lattice girder. With the next the single web or T-shape is in every way preferable.

Of two engineers using Cooper's specification, one will require that the unit stresses in details be increased for dead-load strains, and the other will feel that he is being imposed upon if this be done.

Mr Wright. The above examples of what is daily going on are given, not as criticisms, but merely to help the statement that the writer does not believe a general vote in favor of any one specification is to be expected at this time.

The points in the proposed specification which the writer would criticise are:

(1) Limit to 3 ins. the minimum diameter of rollers for spans over 150 ft. in length; and, by preference, segmental rollers shall be used. Increase the allowed bearing per lineal inch to $800 \sqrt{d}$.

(2) Omit the clause prohibiting the use of plate girders without cover plates.

(3) There is no necessity for the requirement that built chords shall be spliced for the full area of the section, at milled ends.

(4) Pin-plates should be so arranged as to properly transfer to the pins the percentage of stress borne by the cover plate and by the angles, secondary angles being used if necessary.

(5) Make the length of the plates on built sections one and one-quarter times the greatest width of the member where they come at pin-ends and one-half this length at other points.

(6) Specify the size of lattice to be used on larger sections, as where angle lattice shall be used, and where double lattice shall replace single bars.

(7) There is no reason why shoe and bed plates on small spans should be milled. The surface left by the rolls is less susceptible to the action of rust than the milled surface, and is also fully as smooth as the surface left by the average bridge milling machine.

(8) Requirements, such as: "Rivets shall completely fill the holes," "Rivets shall be countersunk where necessary," "Finished members shall be free from twists," etc., "Eye-bars shall be straight," "Eye-bars shall be bored at the same temperature," etc., mean nothing, and merely add to the length of the specification. The first sentence under workmanship, "All work shall be first-class," would seem to cover it all. Of 90% of the so-called "shop instructions," of the average specification, brief notes on the drawings are all that the shop men read or even see.

(9) The limits of phosphorus, sulphur, and manganese are needlessly severe.

(10) A few specifications require (theoretically, at least) that all material shall be oiled at the rolling mill. A few inspectors have also earnestly tried to have this done. They have generally failed, the failure being largely due to the opposition of the mill men; particularly at those mills where material is often loaded on the cars while yet too hot to oil or paint. It is true, however, that the mill men quietly submit to requirements costing more, and of not one-tenth the value that such a one would be.

Mr. Wright.

That material should be shipped and lie for weeks under all sorts of weather without protection of any kind, is inexcusable. Most engineers, however, treat the matter with indifference. Perhaps they rely upon the statement that the paint "holds better" if the surface be first thoroughly coated with rust.

The result of the writer's observation is that soft steel rusts much more rapidly than iron, when exposed without protection to the weather. He has assembled a set of specimens taken from angles, channels and flats which have laid exposed to the weather for periods of from two to sixteen years. Some of the specimens were oiled soon after rolling, others a year or more afterward, and some not at all. Among the unoiled specimens the iron ones show almost uniformly less rust than the steel ones. A very casual examination is sufficient to show the importance of keeping iron and steel structures thoroughly painted, and to show also very conclusively that the time to apply the protective coating is before any opportunity has been given for rust to begin its work.

Mr. Worcester.

J. R. Worcester, M. Am. Soc. C. E.—The author's clear and able presentation of the arguments in favor of the use of the Launhardt formula, or some similar one, for proportioning parts, seems to lack one vital link in the chain of reasoning. If, in designing bridges, the engineer were endeavoring to make them fail under a certain load, or at a certain time, or to make all parts fail at the same time when the opportunity occurred, then he could not do better, probably, than to design them in accordance with the knowledge gained from Wöhler's experiments. When he is endeavoring, however, to so proportion the work that it will never fail, or be distorted beyond the "limit of uniform elongation," he is dealing with an entirely different element in the strength of the material.

This "limit of uniform elongation," or "elastic limit," is the real limit of strength of a structure. Beyond this, engineers hope never to go, and a bridge which has reached it is hopelessly beyond its period of usefulness. It is to keep away from this limit that the "factor of safety" is used, and all parts should be designed with a view to being proportionally distant from this point. So far as the writer is aware, neither Wöhler's experiments nor any others have shown that an infinite number of repetitions of a strain less than the elastic limit of the material (strains in opposite directions being added) has the slightest effect upon the elastic limit; and until it is shown, or reasons are given why it should be inferred, that this elastic limit is reduced by fatigue, it does not appear to be proved that in proportioning parts, the engineer should take account of changes in the amount of stress.

There is, however, a logical reason for allowing a greater strain from dead than from live loads, and that is, that, while the former may be exactly known and never exceeded, the latter involves elements of un-

Mr. Worcester. certainty. Not only is the additional effect of impact uncertain, but the amount of the live load itself—especially in railroad bridges—is also very doubtful, when it is considered that a future period of indefinite extent is being provided for.

Acknowledging, therefore, the wisdom of allowing lesser strains from live than from dead loads, the author's method of proportioning parts has many advantages, above all, the consistency with which it applies to details as well as to main members, but its simplicity is largely decreased by the introduction of an arbitrary and somewhat uncertain addition for impact. As an instance of the uncertainty of this addition, take the case of a floor beam of a double-track bridge: Should the same allowance be made for impact, under the specification, as if only one track were carried by the beam? If impact is to be provided for, it is questionable whether the whole addition to the live load cannot be more conveniently and quite as thoroughly provided for in this way, as Mr. C. C. Schneider has done for many years in his specifications.

On the whole, the specifications proposed by the author are very exhaustive and satisfactory. There are, however, some points which may, perhaps, be open to question.

The writer would strike out the clause which reads: "A type of truss shall be used * * * which subjects no member to alternate strains." This would apparently entirely bar out riveted Warren trusses of any kind, which in certain places are exceedingly valuable, if not indispensable.

The author has followed the almost universal practice of requiring transverse bracing at every main panel point of deck bridges. In single-track bridges this requirement may be justified on the ground that it does no harm, although it is not in conformity with the principle of so distributing the metal that the strains in each piece can be accurately calculated, for it introduces indeterminateness in the strains which can be dealt with only by making arbitrary assumptions. In the case of double-track deck bridges, however, intermediate transverse bracing is worse than useless, for the reason that when a single track is loaded and one truss deflects more than the other, a rigid system of intermediate transverse bracing will prevent the trusses from deflecting in a vertical plane, and will throw the track to one side, at the same time making strains in the lateral bracing from purely vertical loads. This will produce an exceedingly unpleasant, if not dangerous, side lurch in a rapidly moving train, and it is often the cause of a continual stretch of the lower lateral rods. Dispensing with the intermediate transverse frames does no harm, for the top laterals and end cross-bracing can easily be made to resist all the lateral force, and the elasticity of the posts to which floor beams are riveted is always sufficient to allow for the unequal deflection of the trusses without disturbing

the connections of the floor. In through bridges this condition is bound to exist, and why not in deck bridges ? Mr. Worcester.

The author has followed the common practice of not considering any part of the web of a plate girder as acting with the flange. The common justification of this is not that the web does not actually act with the flange, but that it is often spliced, and consequently inoperative, and that the metal thus introduced does no harm. The practice is, however, not only needlessly wasteful of material, but often results in making a girder much weaker than it might be made with the same material. If the specification allowed the use of the moment of resistance of continuous webs, providing that joints which are spliced for shear only should not be considered as transferring flange strains, it might prevent the common custom of breaking the webs at the center, or where the flange strain is greatest. It would also favor the use of deep girders where the depth is optional, and the advantage of increased depth in the way of stiffness is of no small consequence. The practice also tends to making open trusses more economical than plate girders, which would otherwise be more desirable.

The prohibition of plate girders made without upper flange plates, though supported by good authority on theoretical grounds, is of doubtful necessity, and often very uneconomical, while it involves much labor in fitting ties to rivet heads, and afterward in renewing the wooden floor.

The specification for web plates is deficient in that it allows $\frac{3}{8}$-in. webs to be used for very heavy shears—provided that flange angles have two rows of rivets, and the same webs, with no wider spacing of stiffeners, must be used where the shear is decreased to a merely nominal figure. While the science of proportioning webs is in a very primitive state at the present time, it certainly is not common sense that the material in the web should not bear some relation to the shear. Again, it surely cannot be intentional to insist on the use of stiffeners wherever the unsupported depth of webs is thirty times the thickness, or for a $\frac{3}{8}$-in. web, where the unsupported depth exceeds $11\frac{1}{4}$ ins. The many railroad bridge stringers, without vestige of stiffeners, now in service, which, so far as the writer's observation goes, have never shown any lack of strength from this cause, would bear witness to the absurdity of such a requirement. The writer suggests the following empirical formula as one which will give a spacing about in accordance with the best practice, and at the same time pay some regard to the intensity of the shear:

$$d = t\,(250 - \sqrt{5.2\ s})$$

in which d = clear distance between stiffeners, in inches, t = thickness of web, and s = shear in web plate per square inch. The following note should then be added:

Mr. Worcester. "In case the distance between stiffeners, indicated by this formula, exceeds one and one-quarter times the clear distance between flange angles, the stiffeners may be omitted."

The maximum pitch of rivets in the line of strain may well be greater than twelve times the thickness of the thinnest external plate connected, where the rows are not far apart transversely to the strain, as in the case of flanges, where the angles each have two gauge lines.

The wisdom of not specifying any lower limit for the ultimate strength of rivet steel is not very apparent. Though the tendency of the mills may be to get too high steel, it would seem to be rather a radical departure to agree to accept as low material as can be produced.

The writer is not aware of any reason why contact surfaces should be painted rather than oiled before being put together. Such surfaces cannot rust enough to do any harm if not protected at all, and certainly a coat of oil would prevent even incipient rusting in such a place. Moreover, it is a custom of shop men, when painting those surfaces which are to be immediately covered up, to daub the paint brush once over one of the surfaces, the chances being that not over 50% of the area is ever touched by the paint. Under these circumstances, oil is much superior to paint, from the fact that when the parts are assembled it will crawl, by means of its own peculiar power, while any pigment will tend to retard this covering power.

The specifications do not appear to contain any provision for initial strains in adjustable members, and perhaps this is unnecessary, though the effect of this initial strain on struts and connections should not be neglected. The writer would suggest the following clause, the wording of which is taken bodily from the specification of J. P. Snow, M. Am. Soc. C. E.:

"Adjustable members shall be proportioned for an initial strain of 10 000 lbs. When the calculated external strain in adjustable diagonals is less than 20 000 lbs., the maximum strain on the member shall be taken as 10 000 lbs. plus one-half the calculated strain. When the calculated strain equals or exceeds 20 000 lbs., it shall be used as the maximum, and the initial strain ignored."

In the case of a panel where the two opposing diagonals are the same in sectional area, and the resisting struts of so much greater section that they are inappreciably shortened by the tension in the diagonals, this condition is practically true; because, when an external force produces an added strain to one diagonal and the bars stretch, the opposing diagonal is shortened by just the same amount, and, as the changes in strain are practically proportional to the changes in length, it follows that half the external strain is resisted by the increased strain in one diagonal, while the other half goes to replace the strain formerly existing in the other. As the strain from external force increases, it finally reaches a point at which all the initial strain in the opposing

diagonal is replaced, and after that is done there is no more need of considering anything more than the external forces. Mr. Worcester.

When the diagonals are not equal in section, the adjustable member, being the smaller, will be less affected by external strains than its more rigid opposite; that is, less than half the external strain might perhaps be added to the initial strain, but the error involved in applying the same rule is slight, and always on the safe side.

F. E. TURNEAURE, Assoc. Am. Soc. C. E.—The logical sequence of the main operations involved in the determination of bridge stresses and of cross-sections, as it appears to the writer, is: *First*, the determination of the dead-load stresses; *second*, the determination of static live-load stresses; *third*, the addition to the latter of an amount estimated to cover effects of vibration and impact, thus determining as closely as may be the actual live-load stresses; *fourth*, the combination of dead-load with actual live-load stresses by means of the Launhardt formula, or in a manner such as recommended by the author, for the purpose of making provision for fatigue; and *fifth*, the selection of a working stress for dead-load stress, or equivalent dead-load stress, which shall be sufficiently below the elastic limit to leave a margin for undetermined secondary stresses, imperfections in material and workmanship, corrosion and possibly for occasional increase of the maximum stress by a rare combination of loads. Mr. Turneaure.

As the writer desires to discuss impact principally, he will touch on that subject first, although this is not the chief topic of the paper.

Impact and Vibration.—In providing for fatigue, or combining live and dead-load stresses for any purpose, it certainly seems proper to use not the live load stresses as determined statically, but the actual stresses to which the members are subjected, if such stresses can be determined. Unfortunately, the existing data on this point are not very extensive, but enough is known to show that the stresses added by impact and vibration are a very considerable percentage of the static live-load stresses, much greater that those recommended as for impact in the author's specifications. Professor Robinson showed by his experiments* that vibration in ordinary spans, up to 150 ft. or more in length, might be expected to frequently equal 15 or 20% of the static deflection, involving, therefore, a corresponding increase of chord stress. In a series of experiments of considerable extent, recently made by the writer,† Professor Robinson's results have been fully corroborated, and, moreover, it has been found that in still shorter spans this percentage becomes much higher, being at least 40% for plate-girder spans of 25 to 50 ft. in length. Furthermore, this excessive vibration is not of rare occurrence, but takes place so frequently as to

* "Vibration of Bridges," by S. W. Robinson, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. xvi, p. 42.

† *Proceedings*, Am. Soc. C. E., Vol. xxiv, November, 1898, p. 783.

Mr. Turneaure.

merit full recognition as an inevitable part of the live-load stress. These additional stresses were found to be due rather to the vibration set up by unbalanced drivers and other causes, than to the suddenness of the application of the live loads, but so long as unbalanced drivers must be used such stresses should be taken into account.

The writer is not prepared to offer an impact formula, but considers that nothing less than 50% should be allowed for girders up to 50-ft. spans, and 20 or 25% for spans from 100 ft. to 200 ft. in length. For the construction of a sliding scale, it is thought that 75% and 25% for the limiting spans given in the impact table of the specifications would be conservative figures. For very long truss spans it is probable that little or no provision need be made in the main members for impact and vibration, as such would, in any case, be small, and furthermore the maximum assumed loading for such long spans occurs very rarely. The velocity of freight trains also, in most situations, would be too small to cause vibration. More data on this subject, are greatly to be desired, but the writer believes that the provision for impact should be liberal, and that it is better to err by making it too large than too small. He is very strongly opposed to the method, followed in some specifications, of entirely ignoring impact as such, and in letting this matter take care of itself by using a fatigue formula which ostensibly deals only with the question of fatigue. Probably in the minds of the writers of such specifications the formula is intended to cover the unknown effect of impact, whatever that may be, leaving for fatigue whatever is left over. It would seem that a much better plan is to keep the subjects of impact and fatigue separate, and to use separate formulas, as is done by the author; for by so doing the subject is kept clear, and it is much easier for designers to use judgment in modifying either formula for special cases, or in changing such formulas from time to time as the knowledge of these questions increases. To use a fatigue formula alone, or to treat the subject as is done in Mr. Cooper's well-known specifications, is, in reality, making a much greater allowance for fatigue, or for a margin of safety, in long spans than in short ones, as the impact and vibration is unquestionably a greater percentage of the live-load stresses in the short spans. This variation seems to the writer to be in the wrong direction.

Fatigue.—The author has proposed a method of making provision for fatigue, which gives results that certainly agree very closely with the experiments, and which would seem to be a very convenient formula for practical use. The writer does not wish to discuss the advantages of this over the Launhardt or other formulas, but is disposed to question the propriety of making provision for fatigue to the extent indicated by the ordinary formula or by the method proposed in the paper. The practical limit, beyond which it is undesirable ever to stress iron or steel, when the stress is always of the same kind, is the

Mr. Turneaure.

elastic limit, for whenever this is exceeded undesirable deformations are produced in the structure which cause, among other things, a new distribution of stress. The margin between the elastic limit and the ultimate strength is certainly very desirable in case of emergencies, and it is possible that in poorly designed structures secondary stresses are high enough to cause the total stresses to be occasionally in excess of the elastic limit over very short distances, without causing noticeable distortion of the structures. In a well-designed bridge, however, such secondary stresses should be kept low, and the details made of equal or greater strength than the main members, so that excessive and unlooked-for stresses will rarely occur. If the actual maximum stresses never exceed the elastic limit, the ultimate strength of the member, and therefore the margin for emergencies, remains unchanged, no matter how often the stresses are repeated, and, in case the maximum stresses were fully known, the writer sees no reason why it would not be safe to utilize the full elastic strength of the metal. Furthermore, any slight excess of stress over the elastic limit which might occur would be rare, and the element of recovery of the metal would diminish the necessity for special provision for fatigue. In practice, such full provision for fatigue has in reality seldom or never been made, since impact and vibration have been either entirely overlooked, or have been insufficiently provided for in those specifications using a fatigue formula. Even in the present paper the author lays a double duty on his fatigue formula in certain cases where (page 149) he adopts the ratio of 2 to 1 for live to dead-load stress, and then considers this sufficient to provide also for increased stresses due to vibration and shocks, but not to "sudden application," for which a separate provision is made.

On account of the limited knowledge of local secondary stresses, and of the possibility of the maximum stresses occasionally exceeding the elastic limit in some detail, it is probably desirable to differentiate to some extent between fixed and variable stresses, even though the latter are estimated at their true value. The writer would suggest 30% or 40% as a reasonable addition to the actual live-load stresses for fatigue. It is, in reality, as much as is now usually allowed in short spans. For long spans there seems to be no reason for increasing this, as the details are likely to be better and the distribution of stress more uniform. Whatever provision is deemed desirable, the proper live-load stress to be considered is plainly the static stress plus the increase due to vibration and impact. The frequency with which this maximum live-load stress is likely to occur is perhaps a proper subject for consideration in adopting a coefficient for fatigue. This method would give, for spans 50 ft. in length, an addition, first to the static live-load stresses, of say, 50% to determine the actual, frequently occurring stresses; then a further addition of, say, 40% of 150% for

Mr. Turneaure. fatigue, giving a dead-load stress equivalent to $150 + 60 = 210\%$ of the static live-load stress. For longer spans the impact would be less, but the percentage for fatigue would remain the same.

For members subjected to a reversal of stress, the limit which must not be exceeded is the repetition limit as determined by tests, this being less than the elastic limit; and it is certainly desirable to have a greater margin between the working stress for such members and the repetition limit than between the working stress for members subjected to but one kind of stress and the elastic limit. However, it is true here as in the other case, that if this repetition limit is not exceeded, the ultimate strength remains unaffected.

In the proposed specifications the stresses due to the centrifugal force appear to be classed as dead-load stresses. This seems to the writer to be incorrect. For bridges on moderate curves, the speed of 60 ft. per second is low, and the horizontal component of the pressure of the train upon the bridge would seem to be as much a part of the live load as is the vertical component. It would seem also that the percentage to be added to this part of the live-load stress for lateral impact and vibration should be fully as great as for the vertical loads.

Mr. Thomson. T. KENNARD THOMSON, M. Am. Soc. C. E.—Why should more formulas be presented until there are some new and reliable experiments for a basis?

Launhardt's experiments cover fatigue, but any formula for proportioning the members of a bridge should also consider impact, and the only basis for such a formula would be an exhaustive set of experiments extending over several years, and on all kinds of bridges. Any engineer who will devote his time to such experiments will earn the lasting gratitude of his fellows.

The empirical formula proposed by the author is taken partly from Mr. Cooper and partly from Mr. Schneider, both of whose methods give good results, but why should they be mixed? A good many engineers waste their energies splitting hairs when they ought to split their strains in two.

On a short line of elevated viaduct, the speaker once had occasion to calculate the difference in cost, caused by using fiber strains of 8 000 and 16 000 lbs. The strain used was 8 000, but if 16 000 had been used, the total cost of the viaduct would have been decreased by only 9 per cent. The writer has seen many cases where it would have been actually cheaper in first cost to have used more steel, on account of the better prices the shops would have given.

The writer is surprised to see that the author advocates making all bidders submit designs and strain sheets, for, in the first place, it necessitates say from five to ten companies doing a lot of work when only one can secure the contract, and, in the second place, it implies

that the engineers of the railroad are not capable of doing what should be their own work. Mr. Thomson.

Again, if the contractors furnish the designs, they skin everything for fear of losing the work, while if the railroad employs a competent bridge engineer to make plans the work is easily controlled, and if the contract is let by the pound, it does not make so much difference who makes the shop plans.

The author adds one more to the long list of engine diagrams, notwithstanding the fact that it is impossible to get one to cover the ground. For instance, an engineer would not be allowed, generally, to design an elevated railroad to carry the author's engine.

MACE MOULTON, M. Am. Soc. C. E.—While it seems obvious that engineers will probably be unable to arrive at any real agreement as to all the clauses in a bridge specification, yet it is apparently always in order to discuss any suggestions aiming at an approach to uniformity in practice. Taking this view of the matter, the writer desires to offer a few comments on the paper. Mr. Moulton.

There is one statement in the paper which brings out a point which has often struck the writer in examining specifications, and that is: "The specifications have been written from the standpoint of the railroad company rather than from that of the manufacturer." Undoubtedly, this spirit in writing a specification is perfectly proper when carried to sufficient length to insure first-class construction within reasonable limits of economy.

Years ago, when the tools and appliances in use were more crude, and when the shops at which such work was done were merely manufactories, headed and controlled in most cases by plain men of business and men of possibly a high grade of mechanical knowledge, it was absolutely necessary that the engineers designing work, and making the specifications therefor, should specify the exact requirements to the uttermost detail of manufacture. This procedure, through long years of construction under the exceedingly diverse ideas of engineers, has at length resulted in the equipment of shops with many special tools, and facilities for turning out almost anything which buyers really require, or think they require, to meet the conditions of use of the structures to be built.

As time has progressed, the manufacturers have thought it prudent to so specialize their business as to be in position to meet all the requirements of the engineers of railroad companies. This has required them to employ talented engineers, who have devoted most of their professional efforts to this specialty, to attend to the details of this class of work. These shop engineers and structural specialists have naturally been constantly striving to lessen the expense of production, and, at the same time, in most cases, to elevate the standard of excellence of their product.

Mr. Moulton. The first-class bridge shops, by this growth toward perfecting their special work, having thoroughly winnowed out the chaff, in the way of odd and unnecessarily complicated details of manufacture, are in a position to make suggestions to the buyers of their product which will in many cases simplify and thus cheapen the product, without in any way injuring the resulting structure.

It was undoubtedly necessary, in the early stages, for specifications to be so written as to force the manufacturers to improve, experiment, specialize and perfect their product. The period has arrived, however, when they know their specialty thoroughly, and the nearer a specification conforms to current shop and mill methods, holding up the standard of excellence properly, and carefully following the work through all stages to eliminate errors and personal equations of the shop, the better and more economical the structures will be. To this end let the railroad companies engage specialists to handle their structural work, under the supervision, of course, of the proper chiefs, but see to it that these specialists are really competent, not only to make out strain sheets and figure out and draw details, but that they have had proper shop training, so that such details will not be expositions of some peculiar ideas of the individuals, but will be up to date and of a character recognized as "good practice," both by the railway companies and the manufacturers.

It is probably within the observation of every bridge shop engineer of experience that many structures have been built by them from drawings and specifications of railroad companies which cost the railroad many per cent. more than necessary owing to want of knowledge on the part of a designer of what a shop could do and what it could not do, except at a greatly increased outlay, and giving a result no better for it than if the structure had been designed with more knowledge or attention to prevalent first-class shop practice. Generally in such cases it has not been policy for the manufacturer to make any comments to the buyer, for obvious reasons. Possibly economy in the salary of the designer, or misplaced confidence, has cost railroad companies many dollars in this way.

The experience of all in the difference between the cost of the steel used in bridges to-day and that of even four or five years ago is a case in point. Before that time experimentation was the order of the day, and, necessarily, engineers who specified work had to push to get the manufacturers to perfect their product. Now, however, they know what they can produce economically, and which will answer the purposes for which it is required perfectly well, and have boldly made known to all what they can and will make at a fair price, and that all fancy requirements must be paid for at an increased figure. Designing engineers, consequently, in justice to their companies or clients, have taken to specifying their steel more in accordance with the manu-

facturers' methods, and are designing their structures to meet the qualities of material so made with much surer knowledge of the uniformity of their material, and with a resulting economy in the cost of the structure. Mr. Moulton.

While a specification by a railroad company must state just what is required and bring out points which particular experience has shown to be necessary, yet it should be broad enough to allow the special knowledge of the manufacturing engineers to be available and thus lead to a saving in cost of work and to getting the best method worked out for meeting whatever special conditions may obtain.

The writer does not see the force of the author's statement that it is impracticable to obtain fixed end conditions in columns. Struts solidly riveted in with rivets enough to take the whole strain would approach the condition of fixed ends pretty closely.

Regarding the proportioning of details for live and dead-load strains, the most prevalent practice of proportioning the joints for the combined live and dead-load strain would err on side of safety, and the writer believes that the present practice in rivet units is based on experiments more nearly approaching live-load than dead-load strains.

As the author truly states, the paint question is very controversial, but when pure oil is put on as a shop coat, and has several weeks to dry, the writer has found that the surface produced nearest resembles that of glass or varnish, and that the difficulties in mixing and applying a pure red lead coating to such a surface are such that the men "experienced in this work" must be trained in some other sphere than this, and be nearer perfection than any bridge men whom it has been his good fortune to engage, to get results as satisfactory as those obtained with paints of a lighter body.

The writer cannot believe that it is best to be so rigid as the author is in his general provision that no member shall be subject to alternate strains. This would be proper in a pin-connected structure, but in a riveted structure it would militate against economy of design in many cases. Possibly the author's meaning is misapprehended, and it should be noted that he provides a method for proportioning members subject to alternate strains.

In obtaining net sections of riveted tension members the author's rule seems rather severe, as in the case of rivets staggered about 45° or less, it would cause all lines of rivets to be deducted.

The writer cannot see why in steel the zigzag section should be taken at three-fourths value. In some riveted bridges this method of treatment would make the bottom chords 10 to 15% heavier, without justification by any experiments which have come to the writer's knowledge.

In specifications for quality of material the author leaves out bending or drifting tests entirely. Surely these should be provided for, if only for confirmation of other results.

Mr. Moulton. In specifying about wrought iron a very fine quality is called for. The writer notes no provision for its use in the structures, *i. e.*, no special unit strains for iron. If only used for fillers, why specify iron of so high a grade? Should it not be stated just what parts may be of iron?

The author specifies that threads cut on steel shall be filleted. If this is necessary on soft steel, should there not be a provision against square corners in re-entrant angle cuts, such as occur where the flange of a channel or beam is blocked off?

Why are open turn-buckles barred out?

Regarding inspection of work in shops, the writer thinks it would pay the railroad company to employ a competent inspector, who would have some other functions than that of notifying the manufacturer of defects, when he felt so disposed. According to the author, it seems that the manufacturer would be forced to either ignore the inspector entirely, as having no authority, or be subject to delays for consultation with the engineer during the various stages of the work.

Doubtless the contractor should not be released from responsibility in case of defects discovered after the work leaves the shop, but unless the engineer were at the shop very frequently work which would never be discovered at the site of the structure might go through undiscovered by an inspector of the grade indicated. It would seem to be better to have a first-class inspector to whom should be delegated the authority to pass materials and work, unless it is the meaning of this clause that the engineer shall go to the shop and pass on all the work before shipment.

Mr. Ricketts. PALMER C. RICKETTS, M. Am. Soc. C. E.—The author has presented a very interesting paper, and, as might be inferred from his varied experience in structural design, his general specifications would insure the construction of safe and durable bridges.

It should be remembered, however, in considering the paper, that there are three reasons why the use of formulas, depending upon the fatigue experiments of the German experimenters, in the design of the pieces of ordinary structures, is not necessarily rational. The pieces used by the experimenters were small; were neither full-sized nor built pieces; and the method of loading was different from that to which ordinary structures are subjected. In a given time the number of applications of the load in the experiments greatly exceeded the number to which ordinary structures are subjected. The safe allowed unit stresses in structures subjected to moving loads are much smaller than any of those which were shown by the experiments to injuriously affect the material used.

It is well known that in the design of full-sized pieces it is in some cases irrational to use the constants obtained from experiments on small specimens, even when the method of loading the specimens is

substantially the same as that to which the full-sized pieces will be subjected. It is well known that rest may remove even an apparent set; and there is no proof that a rest, equal to the length of time between trains on a railroad bridge, does not remove every evidence and effect of strain in the pieces of a structure properly designed without the use of a fatigue formula. Some of Wöhler's experiments show the very great difference in resistance offered by the specimens for a small diminution of the load, even when both loads were comparatively large; very much larger than the safe allowed unit stresses used in the design of ordinary structures. Mr. Ricketts.

All these things should be carefully considered in deciding whether fatigue formulas, founded upon any experiments yet made, should be used in ordinary structural design. It is not maintained that such formulas are wholly irrational, but it cannot be shown that they are more rational than the methods of proportioning in common use, which have been developed, without the use of fatigue formulas, from general knowledge of structural materials and from observation of existing structures.

These remarks refer to the design of structures. The use of fatigue formulas is much more rational in the design of quickly reciprocating parts of machines, such as the side bars of locomotives.

C. E. Fowler, M. Am. Soc. C. E.—The extended use of Launhardt's formula would seem to be evidence that most bridge engineers have become convinced that it represents actual conditions more nearly than does any other method of dimensioning. That the form is the correct one for a formula for unit stresses would seem easily proven from a purely theoretical course of reasoning. Mr. Fowler.

Nearly twelve years ago the writer, then employed as bridge engineer on an important line of railroad, had occasion to decide how high the unit stresses should go under actual loadings in use, before many old bridges on the road should be condemned. It was the custom among many engineers of roads in that locality not to call a halt until 20 000 lbs. per square inch on iron had been reached. This seemed to the writer to be very unwise, and a method for determining the allowable unit stresses was accordingly developed, and published* some time later.

Starting with the premise that pure live load is twice as destructive as pure dead load, proof of which may be found in Rankine or Burr, it will be seen that when a bridge member is subjected to all dead load the unit stress may be allowed to nearly equal the elastic limit without danger, but when it is subjected to all live load the unit stress must not exceed one-half the elastic limit of the material; for while actual live loads may never equal theoretically pure live loads, they may, in some cases, approximate them so closely as to leave only

* *Engineering News*, April 6, 1889, p. 315.

Mr. Fowler. a small margin for safety. Letting Min. B represent the minimum stress on a member and Max. B the maximum stress on the member, then, when the stress is all caused by dead load $\frac{\text{Min. } B}{\text{Max. } B} = 1$, and when the stress is all caused by live load $\frac{\text{Min. } B}{\text{Max. } B} = 0$.

Letting S represent the maximum allowable unit stress and E the elastic limit of the material, then $\frac{E}{2}$ will represent the superior limit of stress when the load is all live, or, as $\frac{\text{Min. } B}{\text{Max. } B} = 0$, the unit stress allowable will be obtained by multiplying $\frac{E}{2}$ by $1 + \frac{\text{Min. } B}{\text{Max. } B}$, or by unity.

When the load is all dead, then $\frac{E}{2}$ must be multiplied by 2, or, as $\frac{\text{Min. } B}{\text{Max. } B}$ in this case equals unity, by $1 + \frac{\text{Min. } B}{\text{Max. } B}$.

In either case $S = \frac{E}{2}\left(1 + \frac{\text{Min. } B}{\text{Max. } B}\right)$, thus giving a rational method with which to replace a dangerous assumption, and a formula identical in form with Launhardt's. The results from its use were very gratifying, indicating unit stresses ranging from 14 000 to 18 000 lbs. per square inch for iron, where it had been customary to allow 20 000 lbs.

That a formula of this kind can be obtained from a purely theoretical foundation, by ordinary algebraic methods, would seem an additional argument in favor of the retention of Launhardt's formula in bridge specifications. With easily constructed tables and a slide rule to facilitate calculations, it is believed also that this is the simplest of all rational methods of dimensioning.

The method given in the specification for the addition of impact is certainly more rational than the common one of making the span length the criterion as to the percentage. Speaking from an extended experience with specifications using impact additions, the writer, however, cannot believe it to be other than a needless encumbrance to calculations. Perhaps when some of the series of experiments which are under way to determine the actual effect of live load are completed, a method may be devised which will almost exactly represent actual conditions. Until this can be done, it is to be regretted that any further modifications of existing modes should be proposed. No one suffers to any great extent except the estimating departments of bridge companies, where sudden changes from one specification to another, repeated many times during the day, are not only confusing, but absolutely disheartening to the wisher for more uniformity, besides being the feature which adds most to the severe labor of estimating.

Mr. Fowler.

The reduction of the commonly accepted value for the coefficient of sliding friction, 20% to 15%, would be advisable when the effect was to decrease the stress, but where the stress would be increased, 20% would more nearly represent actual conditions.

While every engineer may find occasion at times to use rivets in tension, yet, to sanction the practice in a specification is to cause license to become liberty and give unscrupulous designers an excuse for bad details.

With such extended use of concrete for foundations a value should be given for bearing on concrete masonry, and it would seem a small matter to give different values for different kinds of stone.

Under "Details of Design," it is specified that no part of the web shall be counted as flange section. In the large girders now being built for heavy loadings, the number of cover plates to be piled up can be very wisely reduced by carefully designed longitudinal web splices which will develop the strength of one-sixth the web for flange section, and certainly no objection can be found to this with webs of from $\frac{3}{8}$ to $\frac{1}{2}$ inch in thickness.

The provision for fully splicing the flanges of girders is a wise one, as no dependence can be placed on the abutting of plates or angles; for, no matter how carefully they are planed and fitted, they are almost sure to draw apart in riveting.

The method of proportioning tie plates for transferring the stress borne by each segment is the only logical one, and should be adopted in all important specifications.

The requirements as to length of span which shall require rollers have never been correctly stated in any specification. The providing of rollers should not depend alone upon span length, but also upon the weight supported or size of bearing plate, using them when the size of bearing exceeds 200 sq. ins.

Other things being equal, the best designed pin-connected bridge is that in which the eye-bars are packed as nearly parallel to the axis of the bridge as possible, disregarding the requirement to so pack them that the moment on the pin would be a minimum, figuring the pin after the bars were properly packed and using the size so obtained, provided it was reasonable, and only in case it was not, adopting some other arrangement for packing.

Under "Quality of Material," the steel is specified to have an ultimate strength of from 52 000 to 60 000 lbs. per square inch. Not only is this out of the ordinary limits of most mills, and consequently somewhat hard to obtain promptly, but the writer believes that a vote of those interested would show a decided majority in favor of a soft-medium steel, having an ultimate strength of from 55 000 to 65 000 lbs. per square inch.

A feature under the heading "Workmanship," worthy of commen-

Mr. Fowler. dation, is the requirement for the assembling in the shop of adjacent chords and spliced members which are to be field riveted. This will meet with strenuous objection from shop people and will doubtless add to the cost of work; but it will prevent many misfits and much butchery in the field, to the great betterment of the work.

The subject of "Painting" is one about which the last chapter will not be written for many years, and, until then, it behooves the purchaser to see that the material is kept from the weather as much as possible until it shall have received its first coating. This first coating will never adhere perfectly until the mills shall devise some means for removing the scale from steel, or until the bridge shops are provided with facilities for pickling the steel to remove the scale.

While a specification like the author's, with such a large number of commendable features, calls out such an interesting presentation of experience from engineers, it is most sincerely to be hoped that the time is not far distant when the effort will be for reducing, instead of increasing, the number of bridge specifications.

Mr. Snow. J. P. SNOW, M. Am. Soc. C. E.—If proper limits are assumed it makes but little difference in the final results whether the live load is augmented by some multiplier and added to the dead load and a constant unit strain used, or a reasonable impact allowed for, and varying unit strains used. There is undoubtedly a little less work in proportioning the parts of a bridge by the former method than by the latter, but the difference is slight when proper tables and the slide rule are used. In the opinion of the writer, a specification and strain sheet are more intelligible when actual strains plus reasonable impact are shown and varying units used, than when a system like that recommended by the author is used.

It would seem that the impact produced by unbalanced drivers and defective track and wheels, would better be provided for by a certain actual load applied at any one point in the length of the span. The blow from a flat wheel, for instance, will be the same in actual amount on a 10-ft. span as on one of 100 ft. The practice on the Boston and Maine Railroad is to provide for an impulse at any point equal to a static load of 12 500 lbs. in excess of the typical load, when figuring for consolidation engines; and 15 000 lbs. when using the two-axle load. The different schemes of percentages used by various designers approximate this.

The variation of strains should certainly be applied to all connections fully as much as to the main members.

The Launhardt formula, as modified by William Cain, M. Am. Soc. C. E., $a = u\left(1 + \frac{\text{Min.}}{\text{Max.}}\right)$, is intended to cover both impact and fatigue. If identical coefficients are used with this and the 2 to 1 method, the same results will be obtained at the limits of all live load

and all dead load, but at intermediate points, the two methods vary as much as 12 per cent. Mr. Snow.

Wöhler's experiments with values of $\frac{\text{Min.}}{\text{Max.}}$ between the limits of 0 and 1 seem to be altogether too few to build on to such an extent as has been done. When plotted with values of $\frac{\text{Min.}}{\text{Max.}}$ for abscissas and unit strains for ordinates they fit the Cain formula fully as well as the author's curve; showing a tendency to fall in a straight line or in a curve that is convex upward rather than in a curve convex downward, as is the 2 to 1 formula. The points are so few that it seems ridiculous to contend that any particular formula is established by them. The experiments as a whole undoubtedly prove that the internal

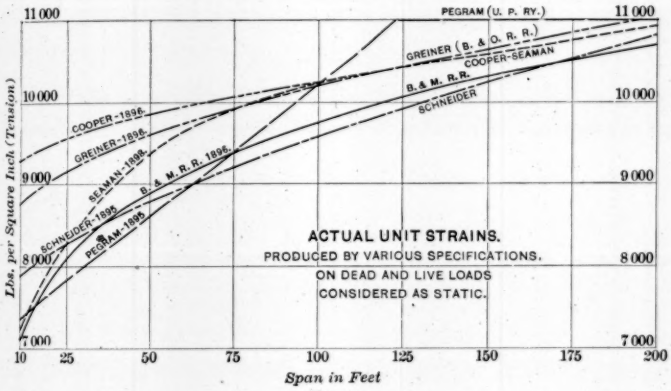


FIG. 4.

structure of material is changed by strain; that the molecules move one on another, and that if the strain is great enough to make this movement continuous each time the load is applied that the piece will ultimately break. It is doubtful, however, if the actual strains usually occurring in good practice will produce this effect. It is likely that a condition of increased elasticity and decreased ductility will be reached that will totally resist all further molecular change under the strain applied. Probably the jar and tremor produced by a moving load in a member when under heavy strain is more effective in producing this rearrangement of molecules than the absolute range from maximum to minimum in ordinary bridges.

To express the effect of jar on a member by means of a formula is altogether beyond the capacity of science at present, and it seems that a reasonable course to pursue is to adopt a scheme of impact adjust-

Mr Snow. ment or unit strain variation, or both, based on actual observation of various types and sizes of bridges. A scheme that produces unsatisfactory bridges is not right even if it is based on some interpretation of Wöhler's experiments.

Perhaps a fair way to compare the practice of different designers regarding impact and fatigue is to assume certain live and dead loads for different span lengths, apply the various rules prescribed for impact and unit strains, divide the modified loads by the allowable strains, and divide the sum of the assumed dead and live loads by this quotient. The result of this division will be the unit strains produced by the actual load considered as static. The diagram, Fig. 4, shows the results of this calculation as applied to the author's recommendations; Cooper's General Specification, 1896; Greiner's Baltimore and Ohio, 1896; Pegram's Union Pacific, 1895; Schneider's Pencoyd, 1895; and the Boston and Maine, 1896.

The assumed loads were obtained from a diagram of weights of iron bridges and corresponding uniform live loads published by G. H. Pegram,* M. Am. Soc. C. E. Class D was used; the equivalent load per foot being $\frac{60\,000}{\text{span}} + 3\,600$. A track load of 400 lbs. per foot was added to the diagram weight of bridge, and the sum taken as the dead load. For the sake of obtaining continuous curves, the unit strain prescribed for eye-bar tension was used for all spans, without considering the change for shapes prescribed by some for the shorter spans.

If the weight of bridges produced by the individual specifications had been deduced, instead of using the diagram weights for all of them, the unit-strain curves would have come somewhat nearer together, but the relative positions would be very nearly as shown.

The writer's experience would lead him to judge that the author's specification will produce disproportionately heavy bridges of 30 ft. span and less, when compared with those of 75 to 125 ft. It would seem that the impact allowance, while being ample for the shorter spans, should be greater for the longer and should extend to those of greater lengths than is proposed.

The rigidity of a bridge is of fully as much moment as its ultimate strength. In recognition of this principle, and to discourage the temptation to use unreasonably shallow beams and girders, the Boston and Maine specification contains the following clause:

"In girders the flange units shall be modified by the ratio of effective depth to length as follows:

"The tension unit shall be $a\left(\frac{1}{2} + \frac{5\,d}{l}\right)$, in which d is the strain depth and l the effective length of the girder. But it shall never exceed $1\frac{1}{4}$ times the value of a."

* *Engineering News*, Vol. xxv, p. 29.

The tension unit corresponding to the ratio of $\frac{\text{Min.}}{\text{Max.}}$ is represented by a. The compression unit is similarly modified. Mr. Snow.

A similar reduction should be used in shallow deck and pony trusses, but an increase for deep trusses would not be advisable, as the chords are light enough without this modification.

A somewhat similar assessment on long, narrow panels of lateral bracing is provided for by the following:

"Tension in laterals shall not exceed 10 000 lbs. per inch; nor $10\,000\frac{2\,d^2}{l^2}$, where d is the distance center to center of trusses and l the length of panel."

It is suggested that something to cover these points would be an improvement to the specification under discussion.

Unless riveted trusses are ruled out altogether by the specification, a point about which the writer is uncertain, it would seem that there should be two formulas for struts. The compression members in the webs of riveted trusses are certainly fixed, and the ends must necessarily remain straight and in the line of strain. A pin may tend to hold a member in this way, and it may not. The top chords of pin trusses should certainly not be considered as fixed, and those of a riveted truss are only partially fixed. It is the custom of the writer to consider partially fixed columns as hinged, with lengths reduced to two-thirds or three-fourths the actual length of the member. The Rankine formula allows too much load on very long, slender columns. It is suggested that some limit or qualifying clause should be used to rectify the formula for high values of $\frac{l}{r}$.

There seems to the writer to be no good reason for neglecting the web in computing the moment of resistance of plate girders. The edge of the web plate must take the same unit strain as the flange, and this must decrease toward the neutral axis. If these conditions obtain, the web is resisting moment and must be carrying load. To say that the web is left free to carry the shear by neglecting to use it for flange is nonsense. The facts of mechanics cannot be changed by assumptions. The claim frequently made, that omitting the web in the flange computation is a measure of safety, is caution thrown away. Plate girders need no special favors when compared with other types of structures, and if figured with all the latitude that can be defended, they will still be better bridges than trusses figured to the same units.

It is not apparent why open turnbuckles for adjustable members should be ruled out. The preference of the writer is exactly opposite to the requirement made in the specification. Sleeve nuts have been known to be split from end to end from the freezing of water which filled the space inside on account of the upper end admitting water

Mr. Snow. while the lower end was tight. Open buckles show the condition of the screw ends much better than closed sleeves, and can be kept from rusting much more effectually.

The author uses soft steel throughout. Is there any objection to higher steel if the work on it is such as not to endanger it? The punch and shear are the enemies of high steel in ordinary bridge work. Eye-bars may well be of medium steel, and pins and rollers of still harder material. Hard steel rollers between hard steel or cast-iron plates may well be loaded higher per lineal inch than allowed by the author.

It is certainly well for engineers to discuss frequently the various points of bridge designing, but a standard specification adopted by so high an authority as a Committee of the American Society of Civil Engineers is probably not desirable. Like some other literature extant it might be made to mean far more than was originally intended. Furthermore, the specification does not design the bridge. After innumerable standard specifications shall have been written it will still need a good designer behind the pencil in order to get a good bridge.

Mr. Waddell. J. A. L. WADDELL, M. Am. Soc. C. E.—The author deserves the hearty thanks of the profession for presenting a paper, the discussion of which will result in one more step toward the attainment of that great *desideratum*, standard specifications for bridges. The writer, however, is of the opinion that the revival of the fatigue theory as applied to bridges is a step backward. The paper makes it very clear that such a thing as fatigue of metal really exists, when the metal is strained above what is commonly termed the elastic limit, but shows no good reason for assuming that it will apply in the case of bridges.

The conditions affecting the metal in the tests and those affecting it in bridges are essentially and fundamentally different. In the former the load is applied every few seconds so that the metal has no chance to recover its equilibrium after one application before the next load comes on, the intensity of stress is above the elastic limit, and the number of applications of the load is very great; while in the latter the loads are applied at comparatively long intervals, even in elevated railroads, so that the metal has a chance to recover its equilibrium between loads, the intensity is only about one-third as great and never half as great as that used in the tests, and the total number of applications of the loading is comparatively small. Such being the case, it appears to the writer to be unsound logic to deduce that what holds true for the metal in the tests will also hold true for the metal in the structures.

That fatigue of metals does exist is recognized even by the uneducated. The writer was struck with this a few weeks ago when fishing for tarpon, with piano wire snells which were rather light. After he had just lost a fish by the breaking of a wire, the boatman, an unedu-

cated foreigner, advised that he use two or three snells, one each day, so as to give the wires a rest, and assured him that they would then break less often. The boatman stated that he had found this out from experience. Mr. Waddell.

In respect to reversing stresses, the writer believes that the time element is a most important factor, or that if the condition passes immediately from tension to compression, then to tension again, etc., the effect will be much more injurious than in the case where the metal has a chance to rest between the applications of opposite stresses.

A glance at pages 142, 143 and 144 shows that, for stresses of one kind, the smallest intensity producing rupture is in excess of the elastic limit, and that for stresses of opposite kinds the sum of the two intensities when rupture is produced is also greater than the elastic limit. There is nothing in Wöhler's experiments to warrant the deduction that every live load is twice as destructive as a dead load. The live load of the experiments was probably just twice as effective as a dead load, in that it was applied suddenly, but the live loads on bridges are not applied suddenly; *i. e.*, it takes an appreciable time to bring them on. For the last twenty-five or thirty years, engineering students have been taught the mathematical demonstration that a suddenly applied load produces exactly twice the extension of a bar in tension that the same load applied very gradually does; and the correctness of the theory has been proved by actual experiment. This idea of double effect, therefore, is no new thing; nevertheless, when it is applied to bridge members, it is, in the writer's opinion, absolutely wrong. Does it not seem evident that if it is proper to apply it to a panel suspender, it would be anything but proper to apply it to the bottom chords of a 500-ft. span? This question is one of actual intensities of working stresses, and can be settled finally only by experiment.

If, for instance, it is known that under the worst possible circumstances the metal in any eye-bar of a bridge is strained actually not to exceed, say, 18 000 lbs. for medium steel, it is taken for granted that it can never be worn out by the loading. Why, then, need engineers trouble themselves to make investigations concerning the least as well as the greatest stress, and complicate the proportioning fully 100%, simply to comply with some experiments that "have nothing to do with the case"?

The simplest, most scientific and most satisfactory method is to add a varying percentage to the greatest live-load stress found on the assumption of static loading, the proper percentage being taken from a table. It is true that the exact values of these percentages are as yet somewhat a matter of conjecture, but some day they certainly will not be. In the writer's opinion, there is no more praiseworthy task for the American Society of Civil Engineers to undertake than to in-

Mr. Waddell. vestigate thoroughly, by numerous experiments and adequate apparatus and appliances, the actual intensities of working stresses for all members of all approved types of modern steel bridges. It would take time and money to do this—the former could be given by the members of the Society, and the latter could be obtained from one or more of America's broad-minded and generous millionaires, provided the matter were properly presented to them.

It seems to the writer that the author complicates designing by using impact and the Launhardt formula combined, and, certainly, his proposed impact allowances are not right, as they reduce to zero for spans exceeding 80 ft. in length.

The paper states that:

"The author is not aware of any specification which has made this distinction between live and dead strains in proportioning details, except those which adopt the Launhardt formula throughout."

The writer published the following in 1898.*

"In spite of all that has been said to the contrary in the past, or that may be said in the future, the impact method of proportioning bridges is the only rational and scientifically practical method of designing, even if the amounts of impact assumed be not absolutely correct; for the method carries the effect of impact into every detail and group of rivets, instead of merely affecting the sections of the main members as do the other methods in common use."

The writer endorses most heartily the author's statement that:

"The details are the most important part of a structure, and those experienced in maintenance will testify that it is the live strain which causes loose rivets and which wears out the bridge. To provide for dead and live strains in the main members, while making no such provision for the details, is to build a scientific structure upon a crude foundation."

This principle was really the true *raison d'être* for the writer's book, before mentioned, and will be found to pervade it from beginning to end.

The writer also endorses most heartily another statement by the author, who, in speaking of inspection says:

"There is no element in bridge construction more important, if properly done, yet none so likely to be neglected. Few appreciate the importance of perfect mechanical work until they maintain the structures they build."

In the specifications proposed by the author it is stated that: "A type of truss shall be used in which the strains may be readily calculated and which subjects no member to alternate strains." Compliance with the latter portion of this specification would involve the abandonment of the Warren girder, which is most useful for deck structures, and rigid diagonals near the middle of truss bridges, and would force designers into using adjustable counters, a type of construction which is rapidly becoming antiquated.

* "De Pontibus," p. 7.

Mr. Waddell.

The distance of 12 ft. 6 ins. between centers of railroad tracks is too small. Most roads require 13 ft., and some of them more.

The reason for not spreading stringers farther apart between centers than their depth is not apparent to the writer.

The writer believes that an assumed wind pressure of 50 lbs. per square foot on an empty bridge is altogether too high, and is entirely unwarranted. All ordinary single-track railroad bridges with eye-bar bottom chords would double up like jack-knives under such a pressure.

It seems unnecessary to proportion the compression flanges of a plate girder by formula; because, if it be stiffened properly by bracing, the same section as that used for the tension flange will be ample.

In specifying 300 lbs. per square inch as the greatest allowable pressure on masonry, the author would load common masonry too high, and would not load the highest classes as much as would be legitimate. Surely there ought to be some distinction in this particular between American natural cement, concrete or brickwork, and granite of the best quality.

On page 159, what is meant by "unequal tearing"? It appears to be an unfortunate term, as any tearing whatsoever would be likely to soon cause the destruction of the bridge.

The writer believes that the web of a plate girder certainly aids the flanges in resisting bending, and therefore ought to be figured on as so doing. If a designer feels confident that this is the case, he is not likely to put in splice plates proportioned only for shear and inadequate to develop the full strength of the web acting as a beam.

The writer takes exception to the clause of the specifications which compels the use of cover plates from end to end on all plate girders. They are an intolerable nuisance to the trackmen on account of the rivet heads. In elevated railroad work cover plates are barred out for this reason. Can the author show good reason, from practical experience in maintenance, for this requirement?

In respect to the stipulation that "all segments of members in compression, connected by strapping only, shall have terminal cross-bracing plates at each end, the rivets and net section of which shall be sufficient to transfer the total maximum strain borne by the segment," the writer would state that once, years ago, when acting as contractors' engineer, and when endeavoring to make, in the office of the consulting engineer, who had inserted a similar clause in his specifications, a design to satisfy the said engineer, he found that in one span the batten plates would not only meet at the middle of the panel, but would have to lap past each other.

Friction rollers $2\frac{1}{2}$ ins. in diameter are rather small. The writer's limit is 3 ins., and it is not unlikely that this also is too low. Rollers of small diameter are likely to rust and fail to act.

Mr. Waddell. It is all very well to specify in respect to workmanship that: "The holes shall be so carefully spaced and punched that, upon assembling, no variation from a truly opposite position of more than one-sixteenth of an inch will occur"; but there is no bridge shop in this country which can comply regularly with this requirement.

The writer's specification* is about as stiff as the shops can well bear.

In the writer's discussion of the paper entitled "The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel,"† by E. Herbert Stone, M. Am. Soc. C. E., he made a comparison of the "nominal stresses" involved by using the impact formula and intensities of "De Pontibus" with those given by Mr. Stone, using as a basis the bottom chords of single-track railroad bridges. As a matter of curiosity, the corresponding "nominal stresses" by the author's method and specifications have been computed, and the results have been recorded in the following table, the author's intensities being increased so as to correspond with medium steel by multiplying them by the ratio of ultimate strengths, viz., 65 : 56.

Span, in feet.	Live load.	Impact.	Dead load.	Nominal intensities per "De Pontibus."	Nominal intensities by the author increased for medium steel.	Percentage of difference.
100.....	4 266	2 844	1 630	12 144	12 121	0.2
200.....	4 030	2 303	2 130	13 102	12 630	3.6
300.....	3 860	1 930	2 820	13 966	13 240	5.2
400.....	3 760	1 671	3 500	14 631	13 765	5.9
500.....	3 700	1 480	4 226	15 169	14 243	6.8
600.....	3 660	1 331	4 930	15 585	14 651	6.0

It will be seen that the author and the writer would not be so very far apart in their intensities of working stresses, if they were using the same kind of steel. However, if the comparison were made with web members, especially where the stresses reverse, the divergence would probably be greater.

There is one point not directly connected with the paper, but brought out by a comment upon it in one of the technical papers, which the writer would like to mention, viz., that in his specifications‡ in spite of the use of an impact percentage, there are three tensile intensities adopted. The explanation of this is, that there ought to be a distinction between steel in eye-bars and the same metal in built tension members, on account of the injury to the latter by punching, etc., in the shop; that there is no real relation between

* "De Pontibus," p. 254.

† *Proceedings*, Am. Soc. C. E., May, 1898, p. 364.

‡ "De Pontibus," Chapter xiv.

the tension in an eye-bar and the extreme fiber stress in a rolled or built beam; and that it did not seem practicable to so adjust the impact formula as to make it apply to both chords and suspenders without using two different intensities of working stress. Consequently the specifications give 18 000 lbs. for chord-bars and eye-bars in diagonals; 16 000 lbs. for shapes in same, for eye-bars in hip-verticals and for adjustable truss members; and 14 000 lbs. for flanges of built beams and for shapes in suspenders. Mr. Waddell.

THEODORE COOPER, M. Am. Soc. C. E.—The remarks made by the writer in the discussion of the paper on "The Kentucky and Indiana Bridge,"* are the views he would now express as applicable to the subject under discussion. Mr. Cooper.

The writer believes that if all the various anomalous ways in which retaining walls stand up or fall down in spite of the theory of earthworks pressures, were classified under the term "fatigue of stone walls" it would be generally considered as "absurd and unscientific." The term "fatigue of metals" is equally absurd and unscientific, and has been a stumbling block to the comprehension of the true principles governing the phenomena classed under this term.

In all branches of science the working theories of the day have had to be rejected, modified or broadened as new phenomena, observed or experimental, have shown the deficiencies of the existing theories.

The theory of the limit of elasticity as heretofore held, and which was based upon experiments commencing at zero and extending to some determined maximum of tension or compression, falls to the ground when the tests, as in more modern experiments, take a more miscellaneous range. It does not follow, however, as the author states, that Wöhler's experiments "entirely destroyed the theory of their (the metals') perfect elasticity," but they do show that the theory as to limit of such perfect elasticity is wrong. Wöhler's tests are in fact the most perfect proofs in existence of the perfect elasticity of steel and iron, within certain determined limits. If a piece of metal remains absolutely unaltered after many millions of applications of strain of various kinds and amounts, could any better demonstration of its perfect elasticity be made?

The author appears to have become confused by the various definitions of the term "elastic limit." That it is necessary and proper for laboratory refinements to "split hairs," need not bother the practical workers in any science. Part of the confusion arises from a misunderstanding. Perfect elasticity and uniform elasticity are not necessarily the same thing. The engineer, from a practical point of view, considers a structure to have perfect elasticity, if under any number of repetitions of the loading it remains unchanged, and returns, after rest, to its original form.

* *Transactions*, Am. Soc. C. E., Vol. xvii, pp. 181-82, October, 1887.

Mr. Cooper. That all test pieces or finished structures may show a slight set has been well recognized from the earliest experiments, and is due to two reasons: *first*, no material is free from internal or initial strains; *second*, the sluggishness of the elasticity; more or less time being required before the piece can return to its original condition.

The writer cannot agree with the author that "this slight set is probably the true explanation of the results obtained by Wöhler."

On the contrary, he thinks that Wöhler's tests show clearly that for practical purposes this small set does not affect the perfect elastic action of the material, when the strains are confined within certain limits.

It is surprising to find the author saying that "many engineers have preferred the concise term, 'Yield Point,' but others still adhere to the old name, 'Elastic Limit,' trusting to a considerate profession to give it the revised interpretation." Does he mean to assert that "yield point" is more concise than "elastic limit," and that those who use the term "elastic limit" are alike confused in regard to these two points? If so the writer desires to go on record in protest. Modern commercial testing is much to blame for this lack of clearness of view. While it does require great refinement of tests to determine with minute accuracy the exact point of the elastic limit, there is no difficulty in determining it near enough for all practical purposes, with ordinary care or ordinary instruments of measurement. The common commercial method of determining it by the drop of the scale beam is absolutely false, and gives errors as high as 10 000 lbs., and always in excess of the true elastic limit.

It is time for those who believe that the "Elastic Limit" is the true test of the capabilities of structural material to point out the falsity of these determinations and insist on the proper methods being used occasionally, as checks upon the cruder and false methods. Any claim made for elastic limits of ordinary structural steel that exceeds 50% of the breaking strength may be looked on with grave suspicion, and should only be accepted after proper methods, carefully worked out, support the claims.

The coincidence, shown in the last column of the table, on page 146, is interesting, and may be more than a mere accident, but the theory formed by the author in explanation is not satisfactory.

If the range of action of the strains is considered, the same figures result, viz.:

First.—The strain passes from zero to — 17 120 lbs., then returns to zero, then to + 17 120, and then back to zero; having passed through a total range of 68 480 lbs.

Second.—The strain passes from zero to — 34 240 and back to zero, or again over a range of 68 480 lbs.

Third.—The strain passes from zero to — 47 080 and back to — 25 680, which also covers a range of 68 480 lbs.

This coincidence does not tend to prove any theory of work, but rather to show the probability that the elastic capabilities of this material are measured by the sum or cycle of strains through which the piece passes, which, in this case, is 68 480 lbs. Mr. Cooper.

ROBERT GILES, M. Am. Soc. C. E.—The writer cannot agree with the author that the experiments quoted form a sufficient basis on which to establish a formula for bridge work. In the bridge work under discussion, it is assumed that the metal is not strained up to the elastic limit. There is a factor of safety to keep it well within that limit, and so long as the metal is not strained up to that point, the theory of the fatigue of metals will not apply. The writer agrees with the author, however, in making the dead-load unit strain twice the live load, and is also thoroughly in accord with him in making provision to proportion the riveting according to both dead and live-load strains. Mr. Giles.

In the specifications, the limit of 5 ft. for the spacing of stringers and deck-plate girders seems to be too narrow, and in standard-gauge track would bring the rails directly over the stringers. A better limit would be 6 ft. 6 ins.

The author, besides rating the live load at twice the dead load, provides for impact, on an arbitrary assumption, and the writer considers that with the proposed unit strains, impact will be sufficiently provided for.

Although providing for certain wind pressures, the author does not state which should be considered as dead or live loads.

In regard to rivets in tension, the author provides that the strains should not exceed one-half of the limit allowed for shear, which may seem a low enough limit; but would it not be better to make no allowance for rivets in tension, and make any details requiring them subject to special approval? Rivets in tension undoubtedly do considerable work, as can be seen in several bridges on the New York Central and Hudson River Railroad, but it would be much better to avoid using them.

The author provides for a single unit stress for the pressure of the rollers. To be logical, this should also be expressed in terms of the dead and live loads.

The minimum limit of 4 ft. between stiffeners in plate girders, specified by the author, would prohibit stiffeners over the bed plates and points of local loadings.

The author's requirements for steel are too low for such pieces as pins, eye-bars, roller bearings and sliding plates. Steel for such members should have an ultimate strength of not less than 64 000 lbs. $\pm$ 4 000 lbs., with an elongation of not less than 25 per cent.

The writer does not understand the clause: "When bolts are used instead of pins, a variation of $\frac{1}{16}$ in. will be allowed between diameter

Mr. Giles. of bolt and hole," when the author has just specified that "no variation will be allowed between diameter of pin and pin hole of more than $\frac{1}{60}$ in."

The writer would suggest that specifications for painting should provide that all painting be done under cover, and that no work be allowed to leave the shops until 48 hours had elapsed from the time of the application of the paint. Bridge work is often loaded on cars while the paint is still green, and the writer has heard of its being painted even on the cars, while they were being hauled out of the bridge company's yard.

In order that there should be satisfactory inspection, and that the railroad company should know what sort of workmanship they are getting, it would seem to be a good thing for all inspectors to report in detail what mill and shop errors are found during the manufacture of the work, and the writer would suggest a clause to that effect.

Mr. Greiner. J. E. GREINER, M. Am. Soc. C. E.—The practice in regard to certain features in bridge design, such as live loads, unit stresses, column formulas and impacts is assuredly in an unsettled condition. This state of affairs is the natural result of a quite prevalent inclination among students, engineers and contractors toward availing themselves of their unquestioned prerogative of compiling specifications whenever they please, and embodying therein such individual fancies as may give the appearance of originality.

No one person, acting alone, can ever hope to bring about any degree of uniformity, so long as this prerogative exists, and so long as the conditions involved are not so much questions of fact, but of opinion, where each individual believes his own *dicta* to be about as good as another's. Engineers will not, therefore, acknowledge any one specification as being so far superior to all others as to be specially worthy of general adoption, and the much-desired uniform practice can be attained, if at all, only through the recommendations of a body of representative engineers who are willing to waive their own individuality for the sake of uniformity. It is now time that a committee of such men should be appointed by this Society, men who have the full confidence of both engineers and contractors, because the existing specifications are of such a variety as to indicate to the ordinary layman an unsettled condition which, to say the least, is not creditable.

An engineer, when compiling specifications, should not lose sight of the fact that two interests, namely, those of the railroad company and those of the manufacturer, are so closely blended that good and economical structures are not likely to be obtained unless both sides are considered. If written from the standpoint of the engineer only, a good bridge may result, but at an excessive cost. If written from

the standpoint of the manufacturer, cheap shop work will probably be the primary consideration and the bridge may lack all other qualities. Substantial and economical structures are demanded by the railroad company, and every specification should aim to embody both of these features. The author states that his specifications are written from the standpoint of the railroad company. Mr. Greiner.

A specification should be explicit. It should not leave to the decision of the contractor such important details as floor beam and stringer connections, the least dimensions of posts and bracing, and the limiting ratio of $\frac{l}{r}$. There are many ways of connecting floor-beams. They can be riveted to posts entirely below the chord pins, or above the chord pins with lower flange above the heads of eye-bars or with lower flange below the heads of eye-bars and on the line with the bottom of posts and connected thereto by means of plates which will furnish a connection for lower laterals. Floor beams also may be suspended by single or by double yoke hangers, by eye-bolts or by plates. In deck bridges they may rest upon the upper chords, or be riveted to posts below the chords. Stringers may rest upon the upper flanges of floor beams, may be riveted to the webs of floor beams, or be both riveted to webs and rest upon substantial brackets. Since all sizes and thicknesses of angles are rolled, the contractor should not be the one to decide whether end-connection angles of stringers and floor beams should be $\frac{3}{8}$ or $\frac{3}{4}$ in. It has been practically demonstrated that some of the above methods are better than others, but the author having given no preference for the best will, therefore, get the cheapest.

The light lateral bracing used a decade ago in spans ranging from 20 to 150 ft. can hardly be considered good and effective for bridges designed for a 50% increase in live load. This bracing is not put into a bridge merely for the purpose of sustaining a wind pressure of 30 lbs. per square foot; in fact, a through-truss bridge would collapse under a light live load in a perfect calm, if the lateral bracing should be taken out. The primary object of lateral bracing is, therefore, to tie the structure together, to resist deformation under the action of the live load, and to eliminate lateral vibration as much as possible. In order to obtain these results satisfactorily, lateral bracing should bear some relation to the parts which it holds together, and should be somewhat more substantial in a bridge carrying 138-ton engines than in one carrying 90-ton engines. The theorist cannot estimate the amount of material necessary for these primary conditions, nor can the manufacturer realize what they are, as he seldom sees his output in actual service. The least amount of bracing to be used in any case is, therefore, a matter of judgment based upon field observation, and every specification which aims at good results should fix definitely the

Mr. Greiner.

least amount of bracing required, no matter whether the wind blows with a pressure of 30 lbs. per square foot or not.

Some provision should be made for elevating the outer rail on curves, as this can be done in a variety of ways, some of which are good, but expensive, while others are vicious, but cheap. The contractor should not have the privilege of inserting a rectangular raising piece under the outer rail, which he will probably do unless the engineer specifies otherwise.

A specification should be consistent. If some of the best types of bridge trusses are to be barred out because they have members subjected to alternate stresses, there is no necessity for describing in detail the manner in which the sections of such members are to be determined. If a unit stress of 1 400 lbs. is permitted in cross-ties resting upon stringers spaced 7 ft. apart, why should this stress be reduced to 1 000 lbs. when the stringers are a few inches farther apart? As the writer understands, operating machinery, including gearing and wedges, is a part of a certain type of bridge, and is usually made of cast steel. Why, then, should it be specified that the structure shall be wholly of rolled steel and wrought iron; and, if such a structure must be made wholly of rolled steel and wrought iron, why specify the quality of cast iron? If the specifications are for "steel railroad bridges" as given in the title, what is the use of describing the character of the wrought iron to be used, especially when no provisions are made for unit stresses?

There should be no ambiguity or uncertainty concerning column formulas. The author gives the formula

$$P = p\left(1 + \frac{l^2}{18\,000\,r^2}\right)$$

for finding the increased stress in a column, due to flexure, in cases where the member is subjected to alternate tension and compression, and states that this increased stress is to be added to the tensile stress, the resulting unit stress not to exceed that specified for live load. It is not positively clear whether he intends to use a tensile or compressive unit stress, as both are specified for live stresses. If the ordinary column formula

$$P = \frac{18\,000}{1 + \frac{l^2}{18\,000\,r^2}}$$

is sufficient to provide for compression alone, and the ordinary unit stress for tension alone, why should it be necessary to increase the compression stress when the column is also subjected to tension? The maximum tension makes the maximum compression neither more nor less, and the writer would prefer to proportion the area for each kind of stress separately and then add the two areas thus found for the required area of the sections.

There are provisions in these specifications quite at variance with the writer's practice, but as they are mainly questions of opinion, discussion would be useless. Personally, the writer has more confidence in physical tests than in chemical analyses, and believes that a better knowledge of the quality of the steel can be obtained by punching, drifting, cold, hot and quench-bending tests than by the most careful chemical analysis. The analyses indicate physical properties not always obtained, while severe physical tests demonstrate practically whether or not the metal is good, indifferent or bad. The writer's practice is to limit the phosphorus only and to rely on the physical tests for defects. Mr. Greiner.

EDGAR MARBURG, M. Am. Soc. C. E.—This paper opens the way once more to a discussion of the conclusions to be drawn from Wöhler's famous experiments and later ones of a like general character. The subject has been presented, however, from a somewhat novel point of view, in that the author attempts to account, on rational grounds, for certain coincidences which, so far as they have been observed at all, seem to have been regarded heretofore as purely accidental. Mr. Marburg.

In order to first determine independently how nearly the approximate constancy of what the author terms the "equivalent dead load" [Min. + 2 (Max. — Min)] might be borne out by Wöhler's results, the writer was led to undertake a careful examination of all accessible literature on the subject. In this examination a number of important errors and disagreements were discovered, which the writer thinks may well be brought to more general notice. They are, in fact, for the most part, pertinent to this discussion.

Wöhler's summary of stress limits, within which rupture was not produced after a vast number of repetitions, as quoted by Bauschinger,* is as follows, in terms of centners per German square inch:

		Max. stress.	Min. stress.
Wrought Iron.—	I. Phœnix axle iron, supplied in 1857.	+160	—160
		+300	0
		+440	+240
Cast Steel.—	II. Untempered spring steel.....	+500	0
		+700	+250
		+800	+400
		+900	+600
	III. Krupp axle steel supplied in 1862	+280	—280
		+480	0
		+800	+350

With the exception of the test on wrought iron, between the limits of +440 and +240, these values were obtained from bending tests, the modulus of rupture being computed by the common formula for flexure.

* "Bauschinger's Communications" (1886), Vol. xiii, p. 44.

Mr. Marburg. The above quantities are given also by Spangenberg,* with the addition of the following shearing values, computed from torsion tests on cast steel, by the common formula for torsion:

	Max. stress.	Min. stress.
IV. Krupp axle steel (cast)	+220	−220
	+380	0

In these last two tests, failure did not ensue after 23 850 000 and 19 100 000 repetitions, respectively.

The foregoing values, reduced to pounds per English square inch, are collected in Table No. 3. In the last column the values given in *Engineering*† for corresponding tests, as quoted in part by the author, are shown.

TABLE No. 3.

Material.	Group.	Stresses.		Values from *Engineering*.	
		Maximum.	Minimum.	Maximum.	Minimum.
Wrought iron	I	+16 600	−16 600	+17 120	−17 120
		+31 200	0	+35 310	0
		+45 700	−25 000	+47 080	−25 680
Spring steel	II	+52 000	0	+53 500	0
		+72 800	+26 000	+74 900	+26 750
		+83 200	+41 600	+85 600	+42 800
		+93 600	+62 400	+96 300	+64 200
Axle steel	III	+29 100	−29 100	+29 960	−29 960
		+49 900	0	+51 360	0
		+83 200	+36 400	+85 600	+37 450
Axle steel	IV	+22 900	−22 900	+23 540	−23 540
		+39 500	0	+40 660	0

The value 35 310 for wrought iron, as given in *Engineering*, is apparently a typographical error, as will be shown presently. All other values from that journal seem to be about 3% in error.‡ This observation applies also to Unwin's tables.§

* "The Fatigue of Metals," pp. 13 and 14.

† Vol. xi (1871), p. 397.

‡ In *Engineering* (p. 200) the value of the German centner is placed at 113 436 lbs. English, for which the writer can find no authority. Its true value seems to be 110.23 lbs. English (50 kg.), as given in the Table of Weights of the "Standard Dictionary," and in the English translation of Spangenberg's "Fatigue of Metals" (p. 12). The metric pound (500 grams) has been the commercial standard of weight throughout Germany only since 1872, whereas Wöhler's experiments were made at Frankfort between the years 1859 and 1870. However, the metric pound has been the legal standard for customs since 1840, was adopted by the railroads in 1851, and its use became more and more general until 1872, when it became the universal standard throughout the empire. The old Prussian pound was equivalent to 467.7 grams (Johnson's and Brockhaus' Encyclopædias). While 110.23 lbs. appears to be the probable value of the centner quoted in these tests, the question here is one of relative rather than absolute values.

The German inch is equivalent to 1.0297 English inches.

§ "The Testing of Materials of Construction." Unwin's values (in gross tons per English square inch) seem to have been taken directly from *Engineering*. The wrong value (35 310) for wrought iron appears also in Unwin.

Mr. Marburg.

The value 35 310, which has been designated a typographical error, is the quantity concerning whose origin the author expresses some doubt. That it is a misprint is attested, not only by the agreement of the corresponding values (31 200) quoted by Bauschinger and Spangenberg, but this value is given also by Weyrauch.* Moreover, the error is indicated by the context in *Engineering*, in which Wöhler's reason for adopting 31 200 (given as 32 100) is set forth. For precisely the same reason the value 34 240, assumed instead by the author, appears inadmissible, namely, that the tension tests resulted in rupture under this load. Wöhler's value was derived from the bending tests on the same material.

The quantities in Table No. 3 are all correctly given by Professor S. W. Robinson, M. Am. Soc. C. E.,† with the exception of the third value for wrought iron, which he placed at 41 600 (400 centners) instead of 45 700 (440 centners). This error is apparently attributable to a misprint in the translation from Spangenberg.‡

Concerning the strength of the three materials named, under static loads, there appears to be considerable uncertainty. Bauschinger, quoting directly from Wöhler, places the average tensile value at 445 centners (range 440 to 450), or 46 300 lbs. It is not clear how this can be reconciled with the higher initial stress of 49 900 lbs. per square inch (480 centners), repeated 800 times before rupture. Presumably the great rapidity with which the stresses were alternately developed and released did not allow a sufficient time interval for rupture at a single application, even for loads somewhat in excess of the static strength. The static tensile strength of the steel axles, Bauschinger places at 1 040 centners, or 108 100 lbs., and states that he is unable to find this factor for the spring steel in Wöhler's publications.

Weyrauch quotes for static tests on Phœnix iron, 57 200 lbs. (4 020 kg. per square centimeter) in bending, and 46 800 lbs. (3 290 kg. per square centimeter) in tension. The value, 46 800 lbs., agrees with the upper limit (450 centners) given by Bauschinger. The considerably higher value found in bending may be attributed to the inaccuracy of the common formula of flexure for stresses exceeding the elastic limit. The static strength of the axle steel, Weyrauch places at 104 400 lbs.§ (7 340 kg. per square centimeter).

Concerning the static strength of the spring steel, Weyrauch writes: "Wöhler found for this steel in (static) bending tests $t = 1\,100$ centners (114 400 lbs.)."

In *Engineering* the following are given as the experimental values in tension under static loads:

* Given as 2 195 kg. per square centimeter, "Iron and Steel Constructions," pp. 43 and 44.

† *Transactions*, Am. Soc. C. E., June, 1886, Vol. xv., p. 439.

‡ The correct value, 440 centners, appears on pp. 13 and 43, but, though a misprint, it is given as 400 centners on p. 41.

§ Whether for tension or bending does not appear. The former seems probable.

Mr. Marburg. Phœnix axle iron (supplied in 1857).. { 47 080 / 48 150 } average, 47 615 lbs.

Krupp axle steel (supplied in 1862).. { 109 675 / 112 350 / 112 350 } average, 111 460 lbs.

These averages, corrected for the apparent error of about 3%, previously mentioned, become 46 200 and 108 200; or essentially the same as given by Bauschinger.

From the above data, Table No. 4 may be constructed.

TABLE No. 4.—Static Strength from Wöhler's Tests, in Pounds Per Square Inch.

Authors.	Group I.—Axle Iron.		Group II.—Spring Steel.		Groups III and IV.—Axle Steel.	
	Tension.	Bending.	Tension.	Bending.	Tension.	Bending.
Bauschinger.....	46 300	.....	.....	.....	108 100	.....
Weyrauch.....	46 800	57 200	.....	114 400	104 400	.....
Engineering.....	46 200	.....	.....	.....	108 200	.....
Probable values..	46 300	57 200	.....	114 400	108 100	.....

The values in Table No. 3 and the probable values in Table No. 4 may be combined in Table No. 5, according to the scheme adopted by the author:

TABLE No. 5.

Material.	Group.	Wöhler's Tests. Maximum.	Wöhler's Tests. Minimum.	Live stress. (Max.—Min.)	Dead stress.	Equivalent dead stress. (Dead + 2 live.)
Wrought iron.	I	+ 16 600	− 16 600	33 200	0	* 66 400 (bending).
		+ 31 200	0	31 200	0	* 62 400 "
		+ 45 700	+ 25 000	20 700	25 000	* 66 400 (tension).
		+ 46 300	+ 46 300	0	46 300	46 300 "
		+ 57 200	+ 57 200	0	57 200	57 200 (bending).
Spring steel.	II	+ 52 000	0	52 000	0	*104 000 (bending).
		+ 72 800	+ 26 000	46 800	26 000	*119 600 "
		+ 83 200	+ 41 600	41 600	41 600	*124 800 "
		+ 93 600	+ 62 400	31 200	62 400	*124 800 "
		(+104 000)	(+ 68 600)	(35 400)	(68 600)	(139 400) "
		+114 400	+114 400	0	114 400	114 400 "
Axle steel.	III	+ 29 100	− 29 100	58 200	0	116 400 (bending).
		+ 49 900	0	49 900	0	99 800 "
		+ 83 200	+ 36 400	46 800	36 400	130 000 "
		+108 100	+108 100	0	108 100	108 100 (tension).
Axle steel.	IV	+ 22 900	− 22 900	45 800	0	91 600 (shearing).
		+ 39 500	0	39 500	0	79 000 "
		+108 100	+108 100	0	108 100	{ 85 400 " / {108 100 (tension).

Mr. Marburg.

The fifth set of results for Group II, enclosed in parentheses, corresponds to the author's set No. 2. The reason for its exclusion by Wöhler is not apparent, except that the result seems anomalous, compared with the other tests on this material, and that the number of repetitions (19 673 000) is about half as great as for the remaining four sets. The rejection of the third set of values for wrought iron, between the limits of + 45 700 and + 25 000 would appear better warranted, for in this case there were only 4 000 000 repetitions.

The static strength of the axle steel (108 100) appears only for tension, and is probably lower than for bending. The static strength in shearing for Group IV (85 400) was estimated by multiplying the above value found in tension by 0.79, this being approximately the ratio between the shearing and bending values, as may be seen from a comparison of the first two sets in Groups III and IV. This value is therefore also probably too low.

The values in the last column, distinguished by asterisks, are those given by the author, after correction.

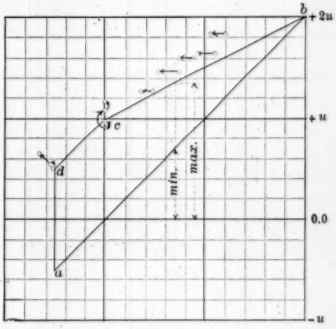


FIG. 5.

From a review of the quantities in the last column, it appears that the experiments give no indication of a law by which the sum, Min. + 2 (Max. — Min.) is even approximately constant for each of the several groups. Although, as the author observes, Wöhler's results for Groups I and II "are more complete and satisfactory than the others," they deserve to be so regarded (especially Group II), by reason of their range, and because the metal in Group I was forged wrought iron. There is no apparent reason, however, for considering the individual results in Groups III and IV as in any way less trustworthy, much less for disregarding them entirely in an inquiry of the kind instituted by the author.

Again, it is to be remembered that Wöhler's results are at best rather loose approximations. The difference between the load by which rupture was not produced after many million applications and the next higher one which caused failure was frequently considerable. The true value is an intermediate one whose magnitude can only be conjectured. The reasoning applied by the author to sets Nos. 1 and 5 is no less applicable to set No. 3.

The whole situation is clearly shown on the diagram, Fig. 5, drawn after the method of Weyrauch. The ordinates from the 0.0 axis to the straight line *a b* represent minimum stresses. The correspond-

Mr. Marburg. ing maxima are laid off along the same ordinates. The experimental values in Table No. 5 are plotted by placing the static strength for each of the several groups equal to $2\,u$. For Group I the static value in bending was used as a basis.* In some cases several values are coincident or sensibly so, of which the diagram gives no indication. The arrows connecting two plotted values indicate the shifted position of the point in question by considering the next higher load which caused failure. The true value lies somewhere between points thus connected. For Group III the data were not available.

To exactly fulfill the law suggested by the author, all points should lie on the line $b\,c$, for stresses of like character, and on $c\,d$ for reversed stresses. The actual departure from such a law is, however, not sufficiently emphasized in this diagram, for, in the formula $c =$ Min. $+ 2$ (Max. $-$ Min.), the true difference between the value of c for any given experiment and the hypothetical constant value $2\,u$ is twice as great as the vertical intercept between $b\,c$ or $c\,d$ and the point which marks the experimental value considered.

However Wöhler's results may be viewed, they admit of no inferences concerning the "theory of work." In the absence of records of deformations, the "work" remains an unknown quantity. It is true that, within the elastic limit, a load suddenly applied, without impact, develops theoretically twice the unit stress produced by the same load gradually applied; but even within the elastic limit this familiar law can find no application here. It is to be remembered that Wöhler's tests were not made by the application and release of known loads, in which case the stresses would have varied with the rapidity of the operations, and would have been different above and below the elastic limit, with other conditions constant. The apparatus was, on the contrary, so adjusted by means of calibrated springs that the stresses themselves remained sensibly constant. This distinction is an exceedingly important one.

Again, there appears to be no ground, in Wöhler's experiments, or any others known to the writer, which would tend to cast doubt, or, as the author asserts, "entirely destroy the theory of the perfect elasticity of metals," it being understood that the accepted theory within the primitive elastic limit is here referred to. This theory found its most triumphant vindication at the hands of Bauschinger himself, whose conclusions the author appears to have somewhat misconstrued. By means of his remarkable measuring apparatus—sensitive to the ten-thousandth part of a millimeter (about $\frac{1}{250000}$ in.)—Bauschinger not only confirmed the correctness of Hooke's classic law, but he showed that within the elastic limit the permanent set, as well as the

*Since this matter has gone to press, the writer's attention has been called to the fact that the static value in tension would have been a more suitable basis, inasmuch as the fiber stresses in bending are within the elastic limit. This would lead to a more marked divergence for this group, from the law suggested by the author, than is shown in the diagram.

total elongation remained constant, both after the lapse of time and under repeated applications of the original load.* Mr. Marburg.

Accordingly, after an initial stress, of a given magnitude within the elastic limit, has been once developed, the material is afterward perfectly elastic up to the limit of that stress; that is to say, complete and instant recovery of form upon release of stress is then assured.

It was, indeed, on these observations that Bauschinger based his claim that stresses within the elastic limit, no matter how frequently repeated, could not be expected to cause rupture. He showed, moreover, that, through the elevation of the elastic limit, this was true also, with certain limitations, for stresses in excess of the primitive elastic limit.† In case, however, the intensity of stress exceeded a certain higher limit, successive repetitions increased the deformation, so that ultimate rupture became then inevitable. The seeming contravention of these laws in the case of reversed stresses has been met by Bauschinger's ingenious theory, recognizing the distinction between the primitive and what he terms the "natural" elastic limit, thus showing that the apparent anomaly is but a paradox.

The author affirms his faith in the "theory of the fatigue of metals," without defining the sense in which this observation is intended. While the signification of "fatigue" is not infrequently misconstrued, and the selection of this particular term may, in the light of present knowledge, have been ill-advised, there appears to be now no reasonable ground for differences of opinion concerning the phenomena for which it stands. The matter has, indeed, long since become one of fact rather than theory. Bauschinger's investigations have apparently furnished conclusive proof that the metal in specimens subjected to oft-repeated tensile stresses experiences no sensible change, either in its molecular structure or in its subsequent behavior under static loads.‡ A theory of fatigue involving an assumed alteration in the physical characteristics of the metal appears then as chimerical as the time-worn theory of cold crystallization. The writer does not wish to be understood as inferring that the author holds any such views, but that they are yet all too prevalent among engineers will probably be admitted. On the other hand, a denial of the reality of such a thing as "fatigue"—understood simply as meaning that rupture will be produced under certain conditions by stresses far within the static strength of a given material—no longer deserves to be entertained seriously.

Returning now to Wöhler's tests, the chief interest attached to them, in the opinion of the writer, is to be found in the fact that they

* "Bauschinger's Communications" (1886), Vol. xiii, pp. 14 and 15.

† *Ibid*, pp. 16 and 39.

‡ In general, the static tensile strength was somewhat raised by oft-repeated stresses. The difference, in every case, was relatively small. The endurance tests in bending, made at the Watertown Arsenal, exhibited, however, in some cases considerable irregularity in his respect.

Mr. Marburg. proved (*a*) that innumerably repeated stresses of like character, not exceeding the primitive elastic limit, cannot produce rupture, and (*b*) that the safe limit for equal stresses, opposite in character, is far below the primitive elastic limit. The former discovery served as a welcome reassurance to the users of metal, the latter as a wholesome warning, despite the fact that the trying condition of reversed stresses had, it seems, been recognized intuitively by American engineers before Wöhler's announcement, and provided for by various methods.

The writer is not in sympathy, however, with the efforts that have been made toward the introduction of so-called "fatigue formulas" into specifications for structural work. This practice is believed to be founded on wrong principles. The added labor it involves—in this the writer speaks from a somewhat extended experience in bridge practice—is to be justified only on grounds of superior scientific excellence, which claim, he thinks, cannot be satisfactorily demonstrated. The writer must refrain, however, from elaborating his views on this point, the contemplated limits of this discussion having already been far exceeded.

Mr. Wilson. JOSEPH M. WILSON, M. Am. Soc. C. E.—It is now more than thirty years since the question of specially providing for the effect of live load on a bridge structure, on different conditions from those of dead load, began to receive serious consideration in America, and much longer than that (nearly fifty years), since such effect was first noticed.

Rankine* states that:

"The additional strain arising, whether from the sudden application or swift motion of the load, is sufficiently provided for in practice by the method already so frequently referred to, of making the factor of safety for the traveling part of the load about double the factor of safety for the fixed part."

It was the custom, even then, to use a higher allowable stress in roofs, and such structures, than in bridges.

Fairbairn,† gives quite a series of tables and experiments to determine the effect of impact, vibratory action and long-continued changes of load on wrought-iron girders.

Unwin‡ states that:

"In the experiments of the Royal Commission on Railway Structures in 1849, when a truck weighing 1 120 lbs. was run over a pair of light cast-iron bars at a velocity of 30 miles an hour, the maximum deflection was twice as great as when the same load quietly rested on the bars." "With 2 066 lbs. the dynamical deflection was three times as great as the statical deflection. But the theory deduced from the experiments led Professor Willis to the conclusion that effects of this kind, produced by a rolling load, and alarmingly manifested in experi-

* "Manual of Civil Engineering," Third Edition, 1864, p. 278 (perhaps also in earlier editions).

† "Useful Information for Civil Engineers," Third Series, London, 1866. *Philosophical Transactions*, 1864, p. 311.

‡ "Wrought-Iron Bridges and Roofs," London, 1869.

ments on a small scale, would be so greatly diminished with bridges of large dimensions as to be of comparatively little importance." Mr. Wilson.

It was, therefore, recognized at that early date that short-span bridges demanded more attention in this respect than those of long span. Unwin also states that:

"In experiments on bars loaded and unloaded alternately, they found that if the deflection did not exceed that due to a statical load amounting to one-third of the breaking weight, the bars suffered 10 000 repetitions of load without appreciable injury; but one bar broke after 50 000 repetitions of load; and when the deflection was increased so as to be equivalent to that produced by a dead load of one-half the breaking weight, all the bars broke after a greater or less length of time."

From these various experiments combined with theory and experience, Unwin deduces* that "for practical purposes, a rolling load may be assumed to be equivalent to a dead load of twice its magnitude," and states that:

"(1) The load may be reduced to an equivalent dead load, whose magnitude will be twice the actual live load plus the dead load," * * * taking "a limiting stress suitable to a dead load." Or:

"(2) The stresses may be calculated from the actual load, considered as a dead load, and a variable limiting stress may be adopted, dependent on the ratio of the dead to the live load."

Unwin submits a table, giving such values for the limiting stress as would be equivalent to allowing twice as much metal for a given live load, as for an equal dead load, as follows:

SAFE LIMITS OF STRESS.

Ratio of live load to dead load.	TONS PER SQUARE INCH.	
	Tension.	Compression.
All dead load	7.00	5.50
.25	5.83	4.59
.33	5.60	4.40
.50	5.25	4.12
.66	5.00	3.93
1.00	4.66	3.66
2.00	4.20	3.30
All live load	3.50	2.75

As to the practice at that time, Unwin states:

"Although the method of proportioning bridges just indicated is really the scientific method, it has not hitherto been often followed in practice. The plan adopted has been to calculate the stresses due to the total load as if quiescent, and to proportion all bridges to a stress lying between the limits which have just been assigned for dead and live loads respectively."

The writer worked on this basis for short-span bridges as early as 1870–71, and it appears to have been adopted by many bridge engineers, and has continued in use by some up to the present day.

* Pp. 36–37, same edition.

Mr. Wilson.

Wöhler's published experiments and Launhardt's formula came later, and were accepted by the writer as furnishing a more rational and proper method of solving the question than anything that had as yet been produced. Wöhler's experiments followed the same line, and confirmed the indications of those previously made, adding materially to the experience which, after all, is what must be depended upon in such matters. The writer is not aware of any later experiments or results to disprove them or to suggest different conclusions. The best that can be said of other methods of procedure is that they conform practically, or nearly so, to that of Launhardt.

The first printed specification by the writer, in which the use of Launhardt's formula appeared, was in 1882, and although he had adopted and used it previously, at least as early as 1880, there was no occasion for its earlier publication, as all calculations and detail drawings for bridges were at that time prepared by him in complete shape ready for bidders, and the method of calculation did not enter into the specification.

The author's specification is commendable in that it admits the value of Wöhler's experiments and Launhardt's formula, and is an effort to simplify calculations without departing materially from the same results. His conclusions are certainly very interesting, but, if he will pardon the writer for drawing attention to the matter, the results are precisely the same as produced by the method proposed by Unwin in 1869 [see (1)].

The writer is not prepared, as yet, to abandon the method he has used since 1880, in the adoption of the Launhardt formula, although should anything be suggested, showing substantial evidence of being really better, he would be very glad to give it favorable consideration. What is needed is a thorough and systematic series of experiments, on a liberal scale, conducted by competent engineers with ample facilities and funds, to establish by actual experience a reliable theory of the action of live load and impact. He joins heartily with Mr. Waddell and Mr. Bouscaren* in this matter.

No amount of pure theoretical discussion will determine it; and as for the experience that engineers talk of, how many of them ever see their bridges after they are up? They design them in the shop and that is the end of their experience. The writer is not referring to the railroad engineer, who does have opportunity to examine his bridges and note their action under service, if he is competent in this line of work; but to the shop engineer who seldom gets the benefit of this experience. The writer thoroughly agrees with the author on the subject of bridge engineers connected with the railroad system, and a periodical inspection of bridges by a thoroughly competent, theoretical and practical engineer.

* *Transactions*, Am. Soc. C. E., Vol. xxvi, pp. 182 and 276.

Mr. Wilson.

It may be a curious thing to say, but each bridge, like many other objects in animate and inanimate nature, has its individual peculiarities, and requires to be studied in its action, which can only be accomplished by periodical inspections. The engineer of bridges should be entirely competent to do this, to prepare specifications, make calculations and inspect material, even if he does not actually make the designs, and to act as an expert for the railroad company in all matters connected with his department, including the criticism and reporting on designs and details of bridges that may be submitted in competition by bridge companies.

The writer has never been able to understand why engineers have so much fear of the labor involved in the use of Launhardt's formula. A table of working stresses, such as Table No. 6, would go far toward relieving such apprehension. From this table the required sections are calculated. The details are then designed directly from the sections of the connected members; for instance, the bearing on pins would equal one and one-half times the total section of the member, etc.

For compression members the permissible stress thus found is reduced to allowable working stress by multiplying by the percentage given in Table No. 7 for struts. The form of this table is peculiarly adapted to labor saving. The value of $\frac{l}{d}$ is always quickly found; the value of $\frac{d^2}{r^2}$ is found to be approximately constant for each different shape of cross-section, as shown in the notes attached to the table, although where accurate results are required this value is usually calculated by working out the moment of inertia of the actual section of the member (particularly for upper chords and vertical struts), the value of r^2 being deduced in terms of d^2.

The application of the proper formula by the use of the table then becomes a very simple matter.

The writer heartily agrees with those who would reduce and simplify the labor of calculation, provided equally reliable results are obtained—results that can be depended upon—but he has seen engineers, otherwise trustworthy and competent, fall into grievous trouble by the use of shorthand methods of computation, where a little care would have saved, not only reputation, but also an immense amount of annoyance to superior officers.

The writer notes the author's remarks on the subject of elevated railways and large railway terminal approaches, where the passage of trains is almost continuous, and agrees that such conditions require special attention. Experience is a valuable teacher, and experience in such cases only dates back in America for comparatively a few years. Suburban travel has increased enormously during that time, and bridges designed on first-class specifications, thoroughly able to carry main-line traffic of the heaviest kind, certainly do appear to

Mr. Wilson.

suffer fatigue and wear out under the frequent repetitions of the same load which they receive when placed in such locations as mentioned.

The writer desires to mention here for the benefit of the profession an experience with pin-connected triangular trusses, having alternate compressive and tensile stresses in the same members, and acting under continuous service as here spoken of, which would lead to the conclusion that this type should not be used, at least in such locations. The rapidly occurring alternate stresses on the braces appeared to cause the pins to revolve in place so that they were worn out very rapidly in deep grooves under the bearings. The pins were calculated on the most liberal allowance for shear and bending moments, according to modern requirements, and the result would have been hard to believe had it not been seen. The writer has seen pins in old bridges of the Linville type, of diameters very considerably less than the present specifications would allow, sustaining severe bending stress and certainly much overstrained (this strain being only in one direction), and standing up to their work perfectly well without showing in the least such wear as above noted.

The writer agrees with the author that it is the live strain which causes loose rivets, and which wears out the bridge.

All riveted connections should have plenty of rivets, and they should be tight. This shows itself forcibly on terminal bridges.

FORMULAS FOR WORKING STRESSES (SEE TABLE NO. 6):

Acting in Single Direction.

$$a = u\left(1 + \frac{t-u}{u} \times \frac{\text{Min. } B}{\text{Max. } B}\right)$$

$$\text{when } t = 2u,\ a = u\left(1 + \frac{\text{Min. } B}{\text{Max. } B}\right).$$

$$c = \frac{a}{1 + \frac{l^2}{5\,000\,w^2}},\ (\text{when } \frac{l}{w} = 12).$$

t = stress for all dead load.
u = stress for all live load.

Acting in Opposite Directions Alternately.

$$a = u\left(1 - \frac{u-s}{u} \times \frac{\text{Max. } B'}{\text{Max. } B}\right)$$

$$\text{when } s = \tfrac{1}{2}u,\ a = u\left(1 - \tfrac{1}{2} \times \frac{\text{Max. } B'}{\text{Max. } B}\right).$$

s = stress for equal tension and compression.
u = stress for either tension or compression alone.

The greater stress upon member = Max. B, and the lesser stress (in opposite direction) = Max. B'.

TABLE No. 6.—WORKING STRESSES, IN NET TONS PER SQUARE INCH. Mr. Wilson.

Min. B. / Max. B.	Acting in Single Direction.* Tension. Value of "a." For high-test tension iron for Max. B.	Acting in Single Direction.* Tension. Value of "a." For ordinary tension iron for Max. B.	Acting in Single Direction.* Compression. Value of "a." For compression, one diameter for Max. B.	Acting in Single Direction.* Compression. Value of "c." For compression (12 diameters) upper flanges pl. girder, braced.	Acting in Opposite Directions,* Alternately. Tension. Value of "a." For high-test tension iron for Max. B.	Acting in Opposite Directions,* Alternately. Tension. Value of "a." For ordinary tension iron for Max. B.	Acting in Opposite Directions,* Alternately. Compression. Value of "a." For compression, one diameter for Max. B.	Max. B / Max. B.
All live								
.00	3.750	3.50	3.250	3.162	3.750	3.500	3.250	.00
.02	3.825	3.57	3.315	3.225	3.713	3.465	3.218	.02
.04	3.900	3.64	3.380	3.289	3.675	3.430	3.185	.04
.06	3.975	3.71	3.445	3.352	3.638	3.395	3.153	.06
.08	4.050	3.78	3.510	3.415	3.600	3.360	3.120	.08
.10	4.125	3.85	3.575	3.478	3.563	3.325	3.088	.10
.12	4.200	3.92	3.640	3.542	3.525	3.290	3.055	.12
.14	4.275	3.99	3.705	3.605	3.488	3.255	3.023	.14
.16	4.350	4.06	3.770	3.668	3.450	3.220	2.990	.16
.18	4.425	4.13	3.835	3.731	3.413	3.185	2.958	.18
.20	4.500	4.20	3.900	3.795	3.375	3.150	2.925	.20
.22	4.575	4.27	3.965	3.858	3.338	3.115	2.893	.22
.24	4.650	4.34	4.030	3.921	3.300	3.080	2.860	.24
.26	4.725	4.41	4.095	3.984	3.263	3.045	2.828	.26
.28	4.800	4.48	4.160	4.048	3.225	3.010	2.795	.28
.30	4.875	4.55	4.225	4.111	3.188	2.975	2.763	.30
.32	4.950	4.62	4.290	4.174	3.150	2.940	2.730	.32
.34	5.025	4.69	4.355	4.237	3.113	2.905	2.698	.34
.36	5.100	4.76	4.420	4.301	3.075	2.870	2.665	.36
.38	5.175	4.83	4.485	4.364	3.038	2.835	2.633	.38
.40	5.250	4.90	4.550	4.427	3.000	2.800	2.600	.40
.42	5.325	4.97	4.615	4.490	2.963	2.765	2.568	.42
.44	5.400	5.04	4.680	4.554	2.925	2.730	2.535	.44
.46	5.475	5.11	4.745	4.617	2.888	2.695	2.503	.46
.48	5.550	5.18	4.810	4.680	2.850	2.660	2.470	.48
.50	5.625	5.25	4.875	4.743	2.813	2.625	2.438	.50
.52	5.700	5.32	4.940	4.807	2.775	2.590	2.405	.52
.54	5.775	5.39	5.005	4.870	2.738	2.555	2.373	.54
.56	5.850	5.46	5.070	4.933	2.700	2.520	2.340	.56
.58	5.925	5.53	5.135	4.996	2.663	2.485	2.308	.58
.60	6.000	5.60	5.200	5.060	2.625	2.450	2.275	.60
.62	6.075	5.67	5.265	5.123	2.588	2.415	2.243	.62
.64	6.150	5.74	5.330	5.186	2.550	2.380	2.210	.64
.66	6.225	5.81	5.395	5.249	2.513	2.345	2.178	.66
.68	6.300	5.88	5.460	5.313	2.475	2.310	2.145	.68
.70	6.375	5.95	5.525	5.376	2.438	2.275	2.113	.70
.72	6.450	6.02	5.590	5.439	2.400	2.240	2.080	.72
.74	6.525	6.09	5.655	5.502	2.363	2.205	2.048	.74
.76	6.600	6.16	5.720	5.566	2.325	2.170	2.015	.76
.78	6.675	6.23	5.785	5.629	2.288	2.135	1.983	.78
.80	6.750	6.30	5.850	5.692	2.250	2.100	1.950	.80
.82	6.825	6.37	5.915	5.755	2.213	2.065	1.918	.82
.84	6.900	6.44	5.980	5.819	2.175	2.030	1.885	.84
.86	6.975	6.51	6.045	5.882	2.138	1.995	1.853	.86
.88	7.050	6.58	6.110	5.945	2.100	1.960	1.820	.88
.90	7.125	6.65	6.175	6.008	2.063	1.925	1.788	.90
.92	7.200	6.72	6.240	6.072	2.025	1.890	1.755	.92
.94	7.275	6.79	6.305	6.135	1.988	1.855	1.723	.94
.96	7.350	6.86	6.370	6.198	1.950	1.820	1.690	.96
.98	7.425	6.93	6.435	6.261	1.913	1.785	1.658	.98
All dead								
1.00	7.500	7.00	6.500	6.324	1.875	1.750	1.625	1.00

* For formulas see page 232.

TABLE No. 7—(Continued).

Average Values of $\frac{d^2}{r^2}$		For Light Weight	For Heavy Weight	Average Values of $\frac{d^2}{r^2}$		For Light Weight	For Heavy Weight
	Even Legged Angles.	26.0	26.4		*Solid Cylinder.*	16.0	
	Uneven " "	10.0	10.0		*Hollow " Thick from .01 to .12 Diam.*	9.0	
	" " "	11.7	11.7		*Solid Square.*	12.0	
	Channels—2" to 12".	6.8	6.8		*Hollow " Thick from .01 to .12 Diam.*	6.8	
	" 15".	7.4			*Solid Octagon*	15.13	
	" 2" to 15".	12.7	12.7		*Hollow " Thick from .05 to .08 Diam.*	8.9	
	Deck Beams—4" to 10".	7.1	7.1		*Built Struts, Average Section b=D.*	10.4	
	" " " " "	35.0	35.0		*" " " "*	10.2	
	I Beams—6" to 15".	6.4	6.4		*Channel Chord.*	6.5	8.00
	" " " "	21.6	22.5		*Built "*	7.0	8.00

Select the proper value of $\frac{l}{d}$ in the left-hand column of the table and follow a horizontal line from this until it reaches a vertical column, at the top of which is the proper value of r^2 in terms of d^2, and there will be found at the intersection a percentage-value with which to multiply a, to obtain b.

$$* b = \frac{a}{1 + \frac{l^2}{36\,000\, r^2}} \qquad + b = \frac{a}{1 + \frac{l^2}{24\,000\, r^2}} \qquad \ddagger b = \frac{a}{1 + \frac{l^2}{18\,000\, r^2}}$$

a = permissible stress previously found,
b = allowable working stress per square inch,
l = length of piece in inches, center to center of connections,
r = least radius of gyration of the section, in inches.

TABLE No. 7.—Strength of Wrought-Iron Struts, Pennsylvania Railroad Specification.

	Both Ends Fixed*													One End Hinged.†						Both Ends Hinged.‡									
	Square of least radius of gyration of cross-section (r^2) =													Square of least radius of gyration of cross-section (r^2) =						Square of least radius of gyration of cross-section (r^2) =									
$\frac{l}{d}$	d^2 6	d^2 7	d^2 8	d^2 9	d^2 10	d^2 12	d^2 14	d^2 16	d^2 18	d^2 20	d^2 24	d^2 28	d^2 32	d^2 36	d^2 6	d^2 8	d^2 10	d^2 12	d^2 16	d^2 24	d^2 6	d^2 7	d^2 8	d^2 9	d^2 10	d^2 12	d^2 14	d^2 16	d^2 18	d^2 20	d^2 24	d^2 28	d^2 32	d^2 36
12	.977	.973	.969	.965	.962	.954	.947	.940	.933	.926	.913	.899	.887	.874	.965	.954	.943	.933	.912	.874	.954	.947	.940	.933	.926	.913	.899	.887	.874	.862	.839	.817	.796	.776
13	.973	.969	.964	.959	.956	.946	.939	.929	.922	.915	.899	.884	.869	.855	.959	.947	.934	.922	.899	.855	.946	.939	.929	.922	.915	.899	.884	.869	.855	.842	.816	.792	.769	.747
14	.968	.963	.958	.953	.948	.939	.929	.920	.911	.902	.885	.868	.851	.836	.953	.939	.924	.911	.884	.836	.939	.929	.920	.911	.902	.885	.868	.851	.836	.821	.793	.766	.741	.718
15	.964	.959	.952	.946	.941	.930	.920	.909	.899	.889	.870	.851	.833	.816	.947	.930	.914	.899	.870	.816	.930	.920	.909	.899	.889	.870	.851	.833	.816	.800	.769	.741	.714	.690
16	.959	.952	.946	.940	.934	.921	.909	.898	.887	.876	.854	.833	.815	.796	.940	.921	.904	.887	.854	.796	.921	.909	.898	.887	.876	.854	.833	.815	.796	.779	.746	.715	.687	.661
17	.953	.946	.940	.932	.925	.912	.899	.886	.875	.862	.839	.817	.796	.776	.932	.912	.893	.874	.838	.776	.912	.899	.886	.875	.862	.839	.817	.796	.776	.757	.722	.690	.661	.634
18	.949	.941	.933	.925	.917	.908	.888	.874	.861	.848	.823	.799	.776	.755	.925	.903	.881	.860	.822	.755	.903	.888	.874	.861	.848	.823	.799	.776	.755	.735	.698	.665	.635	.607
19	.942	.934	.925	.917	.908	.892	.877	.862	.847	.833	.806	.781	.758	.735	.917	.893	.869	.847	.806	.735	.892	.877	.862	.847	.833	.806	.781	.758	.735	.714	.675	.640	.609	.581
20	.937	.928	.918	.909	.900	.882	.865	.849	.833	.818	.790	.762	.738	.714	.909	.883	.857	.833	.789	.714	.882	.865	.849	.833	.818	.790	.762	.738	.714	.692	.652	.616	.584	.556
21	.932	.921	.911	.900	.891	.872	.854	.836	.819	.803	.773	.745	.718	.694	.900	.872	.845	.819	.773	.694	.872	.854	.836	.819	.803	.773	.745	.718	.694	.671	.630	.593	.561	.531
22	.926	.913	.903	.892	.881	.861	.841	.822	.805	.789	.756	.726	.699	.674	.892	.861	.832	.806	.756	.674	.861	.841	.822	.805	.789	.756	.726	.699	.674	.650	.608	.571	.537	.508
23	.918	.907	.895	.883	.873	.850	.830	.809	.791	.773	.739	.708	.680	.654	.883	.850	.819	.791	.739	.654	.850	.830	.809	.791	.773	.739	.708	.680	.654	.630	.586	.549	.515	.486
24	.912	.900	.887	.874	.862	.839	.817	.796	.776	.758	.723	.691	.661	.635	.874	.839	.806	.776	.723	.635	.839	.817	.796	.776	.758	.723	.691	.661	.635	.610	.566	.527	.494	.465
25	.905	.892	.879	.864	.852	.827	.804	.783	.762	.742	.706	.673	.643	.615	.865	.828	.793	.762	.706	.615	.827	.804	.783	.762	.742	.706	.673	.643	.615	.590	.545	.507	.474	.444
26	.899	.884	.869	.855	.842	.816	.792	.769	.747	.727	.689	.656	.625	.597	.855	.816	.781	.747	.689	.597	.816	.792	.769	.747	.727	.689	.656	.625	.597	.571	.526	.487	.454	.425
27	.892	.876	.861	.846	.831	.804	.779	.755	.733	.712	.673	.638	.607	.578	.846	.805	.767	.733	.673	.578	.804	.779	.755	.733	.712	.673	.638	.607	.578	.552	.507	.469	.436	.407
28	.884	.868	.852	.836	.821	.792	.766	.742	.718	.697	.657	.621	.589	.561	.836	.793	.754	.718	.657	.561	.792	.766	.742	.718	.697	.657	.621	.589	.561	.534	.489	.451	.418	.389
29	.877	.859	.843	.826	.811	.781	.753	.729	.704	.681	.641	.604	.572	.543	.826	.781	.740	.704	.641	.543	.781	.753	.729	.704	.681	.641	.604	.572	.543	.517	.471	.433	.401	.373
30	.870	.851	.833	.816	.800	.769	.741	.714	.690	.667	.625	.588	.556	.526	.816	.769	.727	.690	.625	.526	.769	.741	.714	.690	.667	.625	.588	.556	.526	.500	.455	.417	.385	.357
32	.854	.834	.815	.796	.778	.746	.715	.687	.661	.638	.594	.556	.524	.494	.796	.746	.701	.661	.594	.494	.746	.715	.687	.661	.638	.594	.556	.524	.494	.468	.423	.386	.355	.328
34	.838	.817	.796	.776	.757	.722	.690	.661	.634	.609	.565	.526	.493	.464	.776	.722	.675	.634	.565	.464	.722	.690	.661	.634	.609	.565	.526	.493	.464	.438	.393	.357	.327	.302
36	.822	.799	.776	.755	.735	.698	.665	.635	.607	.582	.537	.498	.465	.436	.755	.698	.649	.607	.536	.436	.698	.665	.635	.607	.582	.537	.498	.465	.436	.410	.367	.332	.303	.278
38	.806	.781	.758	.735	.714	.675	.640	.609	.581	.555	.510	.471	.438	.409	.735	.675	.624	.581	.509	.409	.675	.640	.609	.581	.555	.510	.471	.438	.409	.384	.342	.308	.280	.257
40	.789	.763	.738	.714	.692	.652	.616	.585	.556	.530	.484	.446	.413	.385	.714	.652	.600	.556	.484	.385	.652	.616	.585	.556	.530	.484	.446	.413	.385	.360	.319	.287	.260	.238
42	.773	.745	.718	.694	.671	.630	.593	.561	.531	.505	.460	.422	.389	.362	.694	.630	.576	.531	.460	.362	.630	.593	.561	.531	.505	.460	.422	.389	.362	.338	.298	.267	.242	.221
44	.756	.727	.699	.674	.650	.608	.570	.538	.508	.482	.437	.399	.368	.341	.674	.608	.554	.508	.436	.341	.608	.570	.538	.508	.482	.437	.399	.368	.341	.317	.279	.249	.225	.205
46	.739	.708	.680	.654	.630	.586	.549	.516	.486	.460	.415	.378	.347	.321	.654	.586	.531	.486	.415	.321	.586	.549	.516	.486	.460	.415	.378	.347	.321	.299	.262	.233	.210	.191
48	.722	.691	.662	.635	.610	.566	.527	.494	.465	.439	.395	.358	.328	.303	.635	.566	.510	.465	.394	.303	.566	.527	.494	.465	.439	.395	.358	.328	.303	.281	.246	.218	.196	.178
52	.689	.656	.625	.597	.571	.526	.487	.454	.425	.400	.357	.322	.294	.270	.597	.526	.470	.425	.357	.270	.526	.487	.454	.425	.400	.357	.322	.294	.270	.250	.217	.192	.172	.156
56	.656	.622	.590	.561	.534	.489	.451	.418	.389	.365	.324	.291	.264	.242	.561	.489	.433	.389	.324	.242	.489	.451	.418	.389	.365	.324	.291	.264	.242	.223	.193	.170	.152	.138
60	.625	.589	.556	.526	.500	.455	.417	.385	.357	.334	.294	.263	.238	.217	.526	.455	.400	.357	.294	.217	.455	.417	.385	.357	.334	.294	.263	.238	.217	.200	.172	.151	.135	.122

The above quantities are the percentage of crushing strength per square inch of cross-section for the different values of $\frac{l}{d}$ and r^2.

Mr. Seaman. HENRY B. SEAMAN, M. Am. Soc C. E.—It was not the purpose of the paper to offer anything strikingly novel, or unusual, but rather to review the experiments upon which the fatigue formulas are based, and after making reasonable deductions from them, to outline a specification which would embody these results in a concise and practical form. It is to be regretted that any one should have misconstrued these deductions as advocating any precise or restricted interpretation of the tests, as it was repeatedly noted that they were too few, and the material too variable, to permit of any such construction. To propose a new formula, even of the form outlined by Professor Marburg, $C = [\text{Min.} + 2\ (\text{Max.} - \text{Min.})]$ would be to imply a definite knowledge which the experiments do not warrant. At best, the 2 to 1 ratio for fatigue is an approximation; but, being so considered, it is a better general interpretation of those results than is any precise formula which can be presented.

The use of the Launhardt formula has long been considered the most refined interpretation of scientific investigation, and has been extensively introduced in general practice. When the review of the experiments upon which this formula was based showed its inconsistency with those tests, and a more careful analysis tended to confirm what had been the best practice of former years, it seemed proper that the matter should be laid before the Society for consideration. One of the strongest endorsements which any method of dimensioning can receive is the fact that its general features have been long in vogue, and if, in addition to this prolonged usage, it may be shown that the most accurate and scientific experiments yet made upon the resistance of materials, serve generally to endorse this same practice, it will be an additional confirmation which should serve to establish the system.

That the Launhardt formula should have received so little defence in the discussion would appear strange, were it not for the fact that no defence can be made. Mr. Wilson's request for further experiments before discarding the formula would indicate that he has not fully reviewed those already presented. The results of these tests alone, destroy the foundation upon which the formula rests, and its utter unreliability is most fully appreciated after reviewing the method of derivation.

The table comparing the results of the formula with those of experiment, as published in the translation quoted, was as follows (in German centners per square inch*):

	0	250	400	600	1 100
Experiment	500	700	800	900	1 100
Formula	500	711	800	900	1 100

It will be seen that in four cases out of five the results agree exactly, and it is this presentation which probably misled Mr. Wilson, and

* To reduce to pounds per square inch, as per *Engineering*, multiply by 107.

many others, to adopt the formula. Since, however, the value of 1 100 is not in Wöhler's tables of duration tests for spring steel, it would be interesting to know by what higher authority it was introduced by Weyrauch. It is also important to know whether 1 100 centners represents the weight at which the material broke, or the maximum load under which it did not break. In either case, the length of time the test piece remained under strain, as compared with the other duration tests, is important. It is evident that if sufficient time is allowed to receive the advantage of judicious cold-drawing, the ultimate strength will be materially increased. The table, corrected and reduced to English equivalents, is shown on page 147. Mr. Seaman.

From an inspection of Set No. 1 of spring steel it is apparent that the value 53 500 lbs. (500 centners) is too low, and that a weight between this and the next higher (64 200 lbs. or 600 centners) would have been sustained indefinitely.

A change in either the value of u (500 centners) or that of t (1 100 centners) would be sufficient to change the results of the Launhardt formula. Since one of these values is undoubtedly erroneous, while the other is very questionable, there seems little excuse for its further consideration. The fact that u (500 centners) is less than one-half t (1 100 centners) is a further indication of abnormal conditions.

The confirmation of this formula would therefore appear very unsatisfactory, but the manner in which the formula was derived is even less convincing. The statement that the working strength is a function of the live strain is the most vague and indefinite conceivable; and to satisfy this and establish the formula by the interpolation of an arbitrary value which will fill certain stated conditions might suggest inductive rather than deductive reasoning.

Although the Launhardt formula fails, the theory of fatigue has met with more general acceptance than was anticipated. The chief objection appears to be, as already suggested, to the use of the word, rather than to the acceptance of the idea. Mr. Cooper, whom the author regarded as the most prominent opponent to this theory, suggests that within a certain "range of action" (which the author would term range of work, "power of work," or endurance), the elastic capabilities of the material may be measured in cycles of strain, and he includes in this cycle the sum of alternate strains. This is precisely the idea which the author intended to convey, although he did not adopt the word "elastic" because it had been formerly used with a different interpretation.

Mr. Schneider more ingeniously adopts directly the definition of the yield point, or Bauschinger's limit of uniform elongation for his new definition of "elastic limit," overlooking the fact that the original theory of perfect elasticity required not only that the material elongate in direct proportion to the amount of applied strain, but also

Mr. Seaman. that it recover completely its original unstrained condition as soon as the applied load is removed.

It is due to the fact that "after rest," as Mr. Cooper has happily expressed it, the material returns "to its original form," that the term fatigue has been used. It is analogous to the condition of animate nature, which is capable of doing a certain amount of work, provided its range of action is not exceeded; and even exceeding this range or limit, it will recover, after rest. The instance of the fishing snells, mentioned by Mr. Waddell, illustrates its general acceptance, and barbers bear similar testimony in regard to their razors.

Professor Johnson's theory that the true explanation of fracture is due to the development of micro-flaws rather than to fatigue might be tenable if the experiments had all been made on homogeneous steel and the results had proved less uniform; but since fibrous wrought iron was used, in which micro-flaws do not develop, and since the results of the tests on spring steel are comparatively uniform, while the development of micro-flaws is generally confined to a high sulphur steel, is more rapid in hard than in soft material, and does not show any uniformity in results, the micro-flaw theory does not appear to be sustained by experiment.

Soon after the publication of the paper, a statement appeared in one of the technical journals, that since the allowable unit strains are within the elastic limit, there is no necessity of considering fatigue. Several discussions have followed the same suggestion. Such a statement violates the fundamental principle of scientific bridge construction. A well-designed bridge must be as uniformly economical as unlimited life and absolute safety will permit, and there is no more reason why the dead strain should be a ratio of the elastic strength than that the live strain should be a ratio of the alternating strength. It is not merely necessary to build a safe bridge. Such a bridge may be constructed by the selection of a uniform allowable strain based solely upon the alternating strength, but it would be a design which no engineer would endorse.

It is true that bridges are not built to fail, but the proper design of bridges has the point of failure constantly in view, not merely for the purpose of avoiding it, but for the purpose of uniform economy throughout. A factor of safety is selected to cover contingencies which may exist. Where the strains are defined and the contingencies are possible defects in material and workmanship, this factor is usually 3, and provides that only one-third of the full area of the member may be relied upon as perfect when compared with the results of a test specimen. While, therefore, the material may receive a nominal strain of 18 000 lbs. or more per square inch, estimated on the total section, it is supposed actually to be strained close to the ultimate strength of some unseen imperfection. It is this critical

Mr. Seaman.

section, always assumed and ever possible, to which the bridge is designed. This is economical bridge construction, and anything short of it is a crude approximation, wastefully expensive in proportion as the light of experiment and reason is hidden, and the impulse of arbitrary and unreasonable precaution or *dictum* is followed. If the factor of safety is assumed too large it should be reduced, but advantage should not be taken of it to confuse other irregularities which may be properly separated and defined.

Furthermore, it is not merely the construction of bridges which must be considered in outlining specifications, but also their maintenance, since it is here that the closest study of existing conditions is required and for the lack of which many efficient bridges have been consigned to the scrap heap. When the overloaded structures of im-

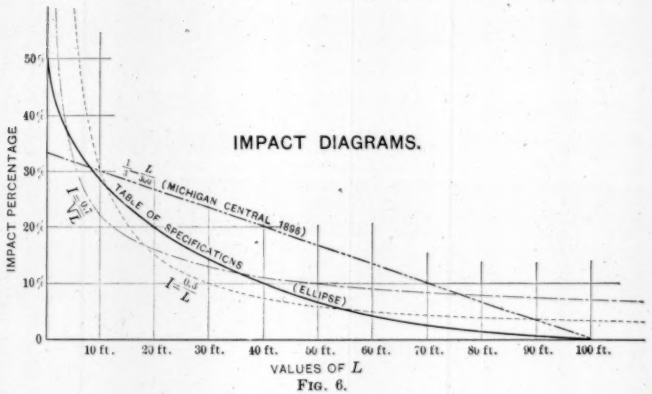


FIG. 6.

poverished roads are to be maintained to the extreme limit of absolute safety, it requires a most careful consideration of the capabilities of the material. The experience of Mr. Fowler in deciding upon the final life of a bridge but illustrates the responsibilities of many others in similar positions. It is a very easy matter to condemn a bridge, but not so easy to maintain it to the final limit. In such cases the question of fatigue is an important one.

The statement that a table for impact complicates an otherwise simple method of dimensioning is, unfortunately, true, but unavoidable. Several years ago, after reading the paper on "Stresses in Bridges,"* by Wm. H. Booth, M. Am. Soc. C. E., the author undertook the theoretical investigation of the effect of impact, upon the assumption that a load instantaneously applied will produce twice the

* *Transactions*, Am. Soc. C. E., Vol. XX, p. 137.

Mr. Seaman. effect of the same load at rest; also that the effect varied inversely as the time consumed in its application. The formula

$$I = \frac{\text{Constant } \sqrt{L}}{L} = \frac{\text{Constant}}{\sqrt{L}}$$

was adopted as a probable representation of the effect of impact. When the present paper was written, the notes upon which this formula was based had been lost, and the formula was, therefore, replaced by the table, the values of which will be found to approximate the curve of an ellipse, as shown in the diagram, Fig. 6.

The formula indicates that impact will be much increased for very short spans and for first panels of long spans. The diagrams of impact taken, both by Professor Robinson and by Professor Turneaure,* confirm this fact as for long spans, and Professor Turneaure's statement that the impact for short spans was so sudden that the instrument would not record the results correctly appears to give confirmation for short spans as well.

The effect of oscillation after the train is fairly on the bridge is given by Professor Robinson† at 50% and by Professor Turneaure as 20 to 30%, with one test on Bridge No. 13 showing 33 per cent. Professor Turneaure also states that the total impact (including vibration) on spans of 50 ft. and less, may be taken at 40 to 50%, and decreases rapidly to, say, 25% for 75-ft. spans, and owing to cumulative effect will remain approximately constant for longer spans (to 150 ft. or more). By adding Professor Turneaure's constant for oscillation (say 30%) to the values of the table, it will be found to give 50% for 20 ft., 40% for 40 ft., 37% for 50 ft., and about 30% for 75 ft. or over. Although impact experiments are extremely rough and approximate, the confirmation is of especial interest. It will be interesting to note what subsequent experiments may show on spans under 20 ft. in length.

The statement that the present specifications were written from the standpoint of the railroad company, rather than from that of the manufacturer, has elicited criticism, but it is not clear upon what such criticism is based. A bridge engineer, with training in bridge shops and experience on maintenance, should be able to weigh the cost and value of various shop requirements and to specify what is most economical to the railroad company, as the purchaser and user, rather than to the manufacturer as the seller. An instance in point is the chemical analysis. The manufacturer's specifications permit 0.08 phosphorus for basic and 0.10 phosphorus for acid steel. The requirements of 0.04 phosphorus for basic is almost universal with railroads and may be obtained without extra cost.

The question of replacing the typical engine of specifications by an equivalent uniform load, with a single concentration, is one which

* *Proceedings*, Am. Soc. C. E., Vol. xxiv, p. 783.

† *Transactions*, Am. Soc. C. E., Vol. xvi, p. 63.

has already been considered* and the reason for not doing so now is, that such a radical change in a general specification should be the work of a committee, if one should be appointed, rather than that of an individual. The proposition appears to be a practical one, and has been adopted by some roads,† but there is a strongly conservative feeling that where the typical engine differs from the equivalent load, the latter conforms more closely to actual conditions, and should be retained. The author believes the difference is so small that the change would be acceptable for new work on account of the greater simplicity. Mr. Seaman.

The reference of Mr. Snow to the importance of making structures rigid as well as strong deserves the greatest emphasis. It is this feature which makes the plate girder the best railroad structure in service, the riveted lattice truss next, and reserves pin-connected work for long spans where the dead load is sufficiently large in proportion to the live to give the structure rigidity. The dead, or initial, strain in any member should be sufficient to overcome any recoil from impact or vibration of the moving load, and where the dead strain is not sufficient, provision should be made for a rigid member or for an initial strain produced by counters. The latter method was successfully used in the Post trusses which were often of short spans, with counter rods in every panel. They were apparently as rigid as the lattice truss, and if the same methods were applied to short Pratt trusses with rigid lower chords, they would probably be found quite as serviceable.

The statement that webs of plate girders resist bending strains is undoubtedly true, and it is partially due to this fact that the plate girder gives such good service. Properly designed web plates which are bulged, however, were probably made so in manufacture, and the suggestion that webs be made sufficiently strong for collision is one provided for by the stiffeners, and which no specification can cover. Although the plate girder is not an articulated structure, the use of stiffeners makes it approximately so, as described by Rankine, the web acting both in tension and compression, the former predominating and the latter existing only to such extent as the tensile strains may stay the web while acting as a column in the diagonally opposite direction. Instances have come to the author's notice where the vertical strain through the stiffener fractured the support, while the strain from the adjacent web, which on account of the flexure of the girder should have been first to act, produced no noticeable effect. Some specifications permit one-sixth of the web section to be estimated as flange area, and it will do this service if it is continuous, but it is not an easy matter to properly splice the web for bending, and with rivets in the vertical splice spaced three diameters apart, it leaves but $\frac{2}{3}$ of

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 229.

† Specifications, Pennsylvania Lines West of Pittsburg.

Mr. Seaman. the web, or $\frac{2}{3} \times \frac{1}{6} = \frac{1}{9}$ of the total section available for equivalent flange section. The clause of the specifications considering the web as resisting shear only, is in very general use and is a conservative provision. In the review of old structures, however, it is proper and customary to make allowance for the resistance of the web.

The placing of stiffeners of plate girders has always been subject to arbitrary specification, and that used in the present instance appears to be the common practice. The suggestion as to double stiffeners at points of support is good, and is not ruled out by the specifications. On the other hand it would be required, whenever necessary, to properly distribute the strain. The proposition to complicate the spacing of stiffeners according to a formula does not seem necessary or advisable.

The explanation of the different formulas in use for finding the allowable strains of rollers, $c \times \sqrt{d}$ and $c \times d$, is very clearly presented by Mr. Morison. The former is based upon the resistance of materials to crushing; the latter is based upon the resistance to the motion of rolling, and is advocated with the statement that the material of bed plates might safely be cold-rolled. This could hardly be accepted to the unlimited degree the formula would permit, and, if carried sufficiently far, would result in the shearing of rivets and general distortion of the metal. Its application seems only justified as an encouragement to the use of large rollers, which feature may be otherwise specified. It would seem preferable, therefore, to adhere to the old form $c \times \sqrt{d}$ and use different values of the constant c for dead and for live load. The latter provision would be for uniformity in the specification rather than for any resulting effect of these loads, since the strains are not received consecutively by the same sections, as the roller moves.

The proposition to specify different allowable strains for different kinds of masonry, such as brick, concrete, etc., implies that this masonry may support the iron bed plate. Such would not be good practice, even for highway bridge work. In every case a proper stone pedestal block or coping is furnished, which will sustain and distribute the applied load of 300 lbs. per square inch. Specifications for the metallic structure should not extend beyond this block.

The proposition to limit the application of the column formula comes from those who have used the straight-line formula, which is of empirical origin and applicable only between limits. The rational formula, however, is not governed by the same limits, but is applicable to long columns. The suggestion that the ends of a riveted lattice web member are fixed is hardly consistent with the statement that the top chords should be considered hinged. If a top chord is hinged, the panel point becomes one of contra-flexure, and this fact would give an initial moment to the web member which would be more injurious

to a column under compression than if the connection were not riveted. Mr. Seaman.

The clause forbidding alternate strains has been very justly criticised, and should be restricted to pin-connected members as originally intended.

It is true that the usual specifications for sliding friction provide for a coefficient of 20%, and this may be obtained under the driving wheels, but the usual friction of stopping trains will not exceed 10%, and the most perfect brake tests on long trains, which have come to the author's notice, showed a coefficient of less than 13 per cent. Since, however, the allowable strain has been raised, it might be advisable to retain the 20% for uniformity. For the same reason the wind pressure should be increased to 40 lbs. to conform to the general practice of building construction. There appears no reason for the distinction which has heretofore existed between these classes of structures, in this respect.

The clause requiring upper-flange plates on all plate girders is chiefly one of æsthetic finish, though it also serves to cover the small water pocket formed in the flange by the web plate and angles. Where there is no upper-flange plate, the web may be made full and no pocket formed, and for deck girders it is decidedly preferable to avoid the varying thickness of flange plates, as well as the rivet heads. It would therefore seem preferable to avoid upper-flange plates entirely, wherever ties rest upon the upper flange; but if any cover plate is used, at least one should extend full length, and all through girders should be so finished. It is not uncommon also to specify round corners on through girders, but as these are particularly difficult to construct, such a clause may be left to the individual preference of the engineer.

The objection to open turnbuckles is principally that they are readily tampered with by the use of a bar. There are few roads which would permit a switch to remain unlocked, and a bridge member would seem almost as important. The author has heard of instances where sleeve-nuts had the lower thread tight, the upper thread loose, became filled with water, froze, and cracked, but has been so unfortunate as never to have seen such an instance. He has always wondered why the lower thread should always remain tight, why it did not become loose after the water froze the first time, and why the material did not stretch a little before cracking. Neither has he been able to understand why a sleeve-nut with tight, compact thread, but with body split by frosty nature, should not be as good as an open turnbuckle, with sides opened in manufacture. The ends of the rods may be located by the punch marks, 12 ins. from the end of each bar, and, if deemed necessary, drain holes may be drilled in the nut.

Mr. Seaman. There does not appear to be unanimity as to whether or not there should be tension on rivets, some preferring no tension, while others claim they will stand full tensile strain. Both are in a measure right. If the rivet holes are perfectly reamed, the edges rounded to $\frac{1}{8}$ in., and rivet heads sufficiently large, there seems to be little reason why a steel rivet should not stand full tension; but when we consider the difficulty of obtaining work of this character, the danger of its neglect, even when specially required, and the impossibility of detecting its omission in the finished work, the use of rivets in tension is a very questionable practice, even though carefully-made tests show favorable results. It seems preferable, therefore, to avoid tension on rivets wherever possible, and in no case to allow more than one-half value.

The suggestion that inspectors should give detailed reports of condemnation, etc., is of very doubtful wisdom. Such a report is interesting and appears to indicate that the inspector's services are valuable, but the custom is very apt to lead to inflated reports and to endless conflict in the shops. The report of an inspection firm which recently came to the author's notice was so full of condemnations and corrections of bad work, that the structure should have been condemned as a whole, and the shops closed. In order to avoid just such artificial reports, the author provides that memoranda may be kept, but no such report made unless called for. By this method, temptation to inflate is avoided, and the engineer receives much closer co-operation in the shops. Only men who are known to be true should be employed, and their work should be checked. In this way a much higher class of work is secured with little friction. It is for the same reason that the responsibility is thrown upon the contractor until the work is completed. The result obtained is the criterion sought.

Mr. Breithaupt's explanation of the distinctive use of the words "stress" and "strain" should be carefully read by all who wish to adopt the former term. The indiscriminate interchange of these words has become very general, even by the best authorities, and it must try the nerves of those who adhere to the distinction, to see such statements as "the material was stressed beyond the elastic limit." The author's preference, however, is to use the word strain in every case. A stress is a delicate force, or tendency to force, which has no place in railroad bridge construction. It should never be used outside of the class-room, and should be restricted there as far as possible. The distinction between applied strain, and internal or molecular strain, may be made when necessary by the proper use of the adjective, and even Rankine, when he suggested the distinction, could hardly have foreseen that incongruous combination "stress sheet," as replacing the good old "strain sheet" of former times.

The question suggested by Professor Merriman as to whether the

percentage of impact be added to the load or to the strain may be answered by considering the purpose of such impact allowance, which is to provide for the increase of applied load due to sudden application. This is directly proportional to the live strain in every instance, even in the case of counter strain, for the live strain is not merely the alternate strain, but it is the total live strain which first neutralizes the initial dead strain, and then, still increasing, produces the alternate strain in the opposite direction. Mr. Seaman.

If Professor Winkler, in the face of Wöhler's experiments, endeavored to disprove the results shown by them, it is not surprising that his task has been a difficult one. If, however, it is their abuse which he deprecates, whether it be an unwarranted precision in their interpretation, or an entire disregard of their teachings, he has led where others must follow.

There is no greater abuse of these experiments than their deliberate misinterpretation, or an endeavor to disparage their results. Precise deductions are not warranted, but to limit their teachings to the statement that there is no fatigue within the elastic limit is to beg the question entirely. To state, as Mr. Schneider does, that "the elastic limit is actually the ultimate strength," is to affirm what these experiments directly disprove. If the "elastic limit" of 34 240 lbs. is the real ultimate strength of fibrous iron, how does the material stand a strain, varying between 47 080 lbs. and 25 680 lbs., repeated indefinitely, without fracture? If the "elastic limit" of 53 500 lbs. is the real ultimate strength of spring steel, how does the material stand, without fracture, strains varying between 74 900 lbs. and 26 750 lbs., between 85 600 lbs. and 42 800 lbs., or between 96 300 lbs. and 64 200 lbs., each in turn repeated indefinitely? The statement "that a single strain (beyond the elastic limit) has practically destroyed the usefulness of the material, and will ultimately produce rupture, if repeated often enough" is absolutely without foundation and contrary to established results.

Table No. 1 as published by Mr. Schneider is interesting, but would have been more valuable if he had described in detail the method and rapidity of loading, so that the difference between these results and those of Wöhler, or those of Mr. James Howard cited by Professor Merriman, might be explained.

The question of impact has already been reviewed, but when Mr. Schneider states that his specifications of the Pencoyd Iron Works for 1887 make a distinction between live and dead strains, he overlooks his former claims that he allowed the same unit strains for live and dead loads, merely increasing the former for impact by formula. He is correct when he states that his provision for impact is far in excess of that shown by experiment. Does not his impact formula also cover fatigue? The author has so regarded it.

Mr. Seaman. Although wind is live strain, it rarely occurs, and the allowable dead strain is permitted. To allow the same strain for a force which is rarely, if ever, applied, as that allowed for one which may be applied at every loading, would be to lose sight of the practical application of the results.

The present tendency toward riveted-lattice trusses for moderate spans is commendable, for the reason that provision is made for reaction after sudden application of the live load, when the live load is large in proportion to the initial dead strain in the member, though there are always objectionable secondary strains. The extent, however, to which the practice of rigid bracing is sometimes carried by manufacturers is ludicrous. It is not an uncommon sight to see the diagonals of trestle bents, or the main laterals of truss bridges, made of angles so long that they sag of their own weight, are of no value as struts, and permit no initial counter strain to give rigidity to the structure.

The reciprocal of the column formula $\left[P = p\left(1 + \frac{l^2}{18\,000r^2}\right)\right]$ will, for alternate strains, replace that formula and not be used in addition to it. This method of dimensioning, therefore, is as simple and direct as that of adding the sections, and is more clearly rational.

The clause requiring six rows of rivets in flanges of plate girders over 16 ins. wide is superfluous and may be omitted. The provision is better covered in a general clause.

The specifications do not require stiffeners on plate girders 24 ins. deep to be spaced closer than 4 ft., unless necessary for concentrated loading.

Terminal cross-bracing plates are properly called batten plates. Tie plates on a railroad are used to prevent rails from cutting into the ties. It is a common practice to make batten plates $1\frac{1}{4}$ times the width of the member.

The first and third clauses by which Mr. Schneider proposes to avoid "gim-crack" construction are good, but if the second clause is used, it should read "resist compression as well as tension," since all lateral systems resist tension, and but few resist any appreciable compression. It should also specify the amount of compression which must be sustained.

The criticism of "lop-sided" unsymmetrical construction will be freely endorsed, and general practice has long avoided it wherever possible, but the plea that bending moments cannot be avoided by placing the pin in the neutral axis is not altogether clear, since the distance from the center of the pin to the center of gravity of the section is a factor of that moment, and reducing this to zero will eliminate any result.

The clause requiring parts to be assembled in the shop is usually

executed by template reaming and should be modified, as Mr. Schneider proposes. Mr. Seaman.

Eye-bars, properly manufactured of soft steel, will not require annealing.

While it may be true in a restricted sense, as Mr. Morison suggests, that columns subject to compression fail by tension, it is not true in the broader interpretation of the condition. A column placed under compression in a testing machine has no other strain applied, and any buckling or apparently tensile fracture may be as truly the result of compression as of tension. There can be no tension developed in any fiber except by a corresponding increase of compression in other fibers, and this increase of compression must be added to that which is received from the end thrust. It is therefore probable that there must be incipient failure by passing the elastic strength in compression, before the buckling takes place, and this, in fact, would cause the buckling. The failure of the column, therefore, is solely and entirely due to compression.

The purpose of allowing different dead and live strains, is threefold: 1st, to provide for any possible effect of fatigue; 2d, to provide for any augmentation of the live load due to vibration; 3d, to provide, to some extent, for possible future increase of moving load over that assumed in calculation. In the first instance it is generally understood that fatigue exists under compression as well as under tension, and requires that alternate strains be added. Also that when the metal is overstrained in one direction, it loses at once its power of resistance in the opposite direction. In the second instance the increased strain due to vibration, and provided for, even by Mr. Morison, in the tension members, is transmitted directly to every compression member. In the third instance any possible future increase of moving load affects every member of the structure, whether in tension or compression. There is, therefore, no reason why a uniform strain should be adopted for compression members while separate strains are used in tension members.

In the statement that plate girders should be made as nearly a solid piece as possible, the author would concur, but he does not share the doubt expressed as to the efficiency of the outer flange plates. No rivets of greater length than five times their diameter should be driven, and holes should be reamed perfect, before driving. If these rivets fill the holes solid there is no reason why they should not transmit strain as well as an equivalent amount of solid material. The practicability of this feature is in the workmanship, and this, in turn, depends upon the specifications and the shop inspection. The greatest increment of strain transmitted to the flange plates occurs at the ends where the thickness of these plates is least. Any strain transmitted near the end is thoroughly distributed over the entire pile by

Mr. Seaman. the time it reaches the center, and the increment at the center of the girder is readily provided for by the large excess of rivets which are always there present.

It is somewhat disappointing to find less corroboration in the Michigan Central specifications than was anticipated from Mr. Schaub's remarks. Those specifications allow live strains at two-thirds the allowable dead strain, and use the impact formula $I = 1\frac{1}{3} - \frac{L}{300}$ if L is less than 100 ft. Other than this there seems to be no confirmation.

The practical application of the 2 to 1 ratio far antedates 1882, as has been so fully cited by Mr. Wilson; but that fact does not detract from the value of Mr. Schaub's endorsement.

Mr. Wright deplores the prospect that machinery, now used for reaming high steel, may become idle upon the advent of the softer material, but it is doubtful if such would be the case. Reaming has been specified for the purpose of removing injured material around the holes, but the matching is often so imperfect that all reaming is on one side of the hole, leaving the other side untouched. Under these circumstances the end sought is not accomplished and a softer steel is advocated. The mismatched holes continue, however, and it is probable that these machines, now used to remove injured material will ultimately be required in all cases to produce perfect workmanship.

The author regrets to have been misunderstood by Mr. Worcester as advocating the use of the Launhardt, or of any other precise fatigue formula. The purpose of the paper was to show that no exact formula was justified by the experiments, but rather that fatigue could be considered in a more general, though only approximate, ratio.

To the inquiry as to whether the same allowance be made for impact in floor beams of a double-track bridge, as would be provided for a single-track, the reply would be the same as for other parts of the structure. If it is possible to load them simultaneously it may also be possible to receive the same impact simultaneously.

The proposition to abandon the intermediate transverse bracing in double-track through bridges has been often urged, but the objections to its use appear to the author unfounded, and the bracing is advantageous in giving compactness to the structure. The side deflection described is inappreciable, and decreases the uneven vertical deflection, thus tending to keep the floor level under partial loading. The increased deflection of one truss beyond the other, which results from the omission of this bracing, tends to increase rather than to decrease any uneven deflection of track. The arguments offered for its omission apply equally well to high through bridges, with overhead sway

frames. Is it also proposed to omit these, and intermediate knee braces as well? Mr. Seaman.

The reason for the omission of the lower limit in the ultimate strength of rivet steel, is that no material which will show the elongation specified can fail to show a satisfactory ultimate strength. There is no objection, however, to its insertion, if preferred.

To the suggestion that oil be used instead of paint for surfaces in contact, because the practice is to paint but one surface, it may be said, that both surfaces should be painted. This is a question of inspection. Oil without pigment is not a permanent protection to iron.

The clause for initial strain in adjustable members, quoted from Mr. Snow, is most excellent, though to those not familiar with its use it might appear to elaborate a somewhat arbitrary requirement.

In Professor Turneaure's suggestion that the specification does not make sufficient provision for impact, he has apparently overlooked the fact that the ratio of 2 to 1, as used in the specification, provides for vibration, etc., as well as for fatigue. What proportion of this ratio is really necessary to provide for fatigue may be a debatable question, but if Professor Turneaure's allowance of 30 to 40% be taken and the difference—say, 60%—be added to the quantities given in the impact table, the allowance will be found to be very liberal.

The suggestion that centrifugal force be treated as a live load, is pertinent, though it should be remembered that this force is entirely dependent upon the speed, which is variable and uncertain, while the vertical loads are dependent upon gravity, which is always present. The bridge being designed for maximum conditions, and the item a small though important one, it would be well to make the provision suggested. Sixty feet per second is a maximum speed for heavy freight trains, for which the bridge is designed.

Of the general importance of familiarity with shop practice by designing engineers there can be no question, but to the attitude of Mr. Moulton that, as compared with engineers employed by railroads on maintenance, shop specialists alone understand the science of bridge design, the author would take most strenuous exception. Experience in shops, contemporary at least in part with that of Mr. Moulton, has taught the author that the best designs which "manufacturing engineers" produce, are those which maintenance has shown to be most serviceable, and which will be found in every case, to conform to some sound principle of mechanics. The statement that manufacturers "know their specialty thoroughly" is reassuring, and if true, there should be no necessity of the shop inspector assuming the functions of a consulting engineer as Mr. Moulton suggests. The assumption that the manufacturing engineer has sufficient knowledge for the inspector's guidance is the very danger to be avoided. It might save the reference

Mr. Seaman. of important questions to headquarters, but could hardly be considered advantageous from the railroad standpoint.

The economies of the past few years are not due to any change in design to conform to particular shop practice, but rather to the financial stringency through which the country has passed during that time. Prices were reduced so low as barely to pay running expenses, in order that bridge companies might continue to exist. It has resulted in cheaper work, but the work has also deteriorated in quality, and it is believed that renewed activity will, to some extent, restore former conditions.

The statement that a specification "should be broad enough to allow the special knowledge of manufacturing engineers to be available," explains why it should embody general principles rather than specific details.

Bending and drifting tests have been omitted because they are crude in comparison with the tests specified, and are valueless either to confirm or to refute those tests. It is very poor material which will not stand the drifting test, carefully made, and such a test is worthless in accepting material, though it may reject it.

The specification for wrought iron is for high-test material to be used where loop-eyes or screw-ends are desired. It is believed that iron is preferable in all such cases, even with the same unit strain, to the best grade of steel.

Square corners in re-entrant angle cuts should be avoided, as Mr. Moulton proposes.

The clause allowing three-fourths value for oblique section between rivet holes is based upon the decreased power of an oblique resistance, rather than upon any similarity to fibrous wrought iron.

The suggestion of Professor Ricketts that rest "equal to the length of time between trains on a railroad bridge" may remove any effect of strain, depends upon the frequency with which trains are run. In this connection the table of Mr. Breithaupt showing that material will stand a greater number of repetitions at the rate of 72 per minute than at the rate of 18 per minute is interesting. If it takes days to remove the effect of strain, the use of the bridge must be greatly restricted. The author has already expressed his concurrence with the view that precise fatigue formulas are not justified, but he believes that fatigue must receive consideration in a structure which may be subject to constant service.

Mr. Fowler's method of deducing safe strains can hardly be considered an independent demonstration. It is applied in the extreme values only, and, therefore, is no more a confirmation of the Launhardt formula than it is of the 2 to 1 ratio. The method which is sometimes used in reviewing old bridges is to assume an extreme value for u in the Launhardt formula and find the corresponding strain allowable.

Mr. Seaman.

The contention of Mr. Snow that Wöhler's experiments are too few to justify any precise formula for fatigue is well founded. They do show that fatigue exists, but the exact effect can only be approximated, because the element of time is an important factor and has not received sufficient consideration in the experiments.

The question whether web members of riveted lattice trusses should be considered fixed or hinged has already been discussed, but the suggestion that upper chords of lattice bridges may be partially fixed while those of pin trusses are certainly hinged, calls attention again to the reason for the distinction. The difference is due to rigidity which may be received from the web members, and this can only be produced by a severe bending strain, which would be more dangerous to web members under compression, than if their ends were free.

The use of high steel for eye-bars seems illusory because the bars are thoroughly annealed, and the material so softened that a milder product might have been specified in advance. It is a moot question whether annealing is not more injurious than beneficial to the material. Hard spots and incipient cracks are more prevalent in hard than in soft material and are especially objectionable under vibratory tensile strain.

The defects incident to high steel would appear to be more dangerous in pins than in any other part of the structure.

The question of fatigue and its application to bridge construction has already been considered in this discussion, but the following statement by Mr. Waddell illustrates the tenacity with which the term "elastic limit" is retained: "For stresses of opposite kind the sum of the two intensities, when rupture is produced, is also greater than the elastic limit." The fact that the alternate strains must be added, to equal the "elastic limit," destroys that term, as formerly accepted. If the material were perfectly elastic up to 34 240 lbs. per square inch, a strain of — 25 000 lbs. could be applied and removed without effect. A reverse strain of + 25 000 lbs. could likewise be applied and removed without effect, and the material could alternate indefinitely from one extreme to the other. Such is not the case, however, as the material stood only 17 120 lbs. under reverse strains. This demonstrates that 34 240 lbs. was not an "elastic limit" but, rather, was its range of action, or power of work, or endurance.

Mr. Waddell appears to be somewhat confused as to the distinction between the effect of fatigue on a 2 to 1 ratio, and the effect of a suddenly applied load which produces twice the strain of the same load at rest. The suggestion "that every live load is twice as destructive as a dead load" is startling, as it would thus require but four repetitions of the allowable live strain to destroy the material. Could the author have been so misunderstood? To the statement that "the live load of the experiment was just twice as effective as a dead load,

Mr. Seaman. in that it was applied suddenly," Professor Marburg's statement is a correct reply:

"It is to be remembered that Wöhler's tests were not made by the application and release of known loads, in which case the stresses would have varied with the rapidity of the operations, and would have been different above and below the elastic limit, with other conditions constant. The apparatus was, on the contrary, so adjusted by means of calibrated springs that the stresses themselves remained sensibly constant. This distinction is an exceedingly important one."

It is unnecessary to add that the Launhardt formula has not been introduced into the specification, as Mr. Waddell states.

The impact effect upon a panel suspender, as compared with that of a chord, has no connection with the question of fatigue.

"Unequal tearing" alludes to the practice of connecting but one leg of an angle in tension, and making no provision for the resulting moment, or tendency to tear, in the piece.

The author has not found opportunity to read "De Pontibus," but if there is anything therein contained which permits a variation in punching exceeding that for which the bridge is designed, it is bad practice, and should be corrected. If engineers are thus lax, what may be anticipated from inspectors?

To Mr. Cooper's discussion the author has little to add. If he prefers the expression "range of action" to "range of work" or "power of work," the preference will not be disputed, and even if the old term, "elastic limit," were completely destroyed in its old meaning and revived in a new one, there might be little objection, although there is no "limit" proper, but rather an elastic strength or power of work. The practical impossibility, however, of completely shaking off the old interpretation is shown by Mr. Cooper's suggestion that occasional tests be made for comparison between it and the "yield point," which is now found in practical testing by "drop of beam."

If the time should ever come when stone masonry is shown to have a definite strength within a certain "range of action," and to require a time of "rest" to restore its original strength, we may well use the expression, "fatigue of stone walls."

Mr. Giles' suggestion that the 2 to 1 ratio provides sufficiently for impact implies that impact would be the same for all spans. All experiments made, however, show conclusively that there is a large excess due to sudden application of the load on short spans and on the first panels of long spans. There appears to be no way of providing for this except by a special impact formula or table.

The clause providing that wind pressure, etc., shall not produce greater strain per square inch than that allowed for dead strain, is based upon the infrequency of the strain rather than upon any supposition that wind, etc., is dead load.

The author wishes to endorse Mr. Giles' suggestion that tension on

rivets should be avoided, but if in exceptional cases it is unavoidable, a prescribed allowable strain should be established. Mr. Seaman.

The clause providing for the use of bolts instead of pins refers to rough bolts, but they might well be omitted altogether on this class of work.

The suggestion of Mr. Greiner that a committee should be appointed for the purpose of procuring greater uniformity in bridge specifications deserves support. There is little danger that the work of such a committee would be misunderstood or considered a bar to individual development, but it would tend to eliminate many useless differences which at present exist, and would record individual views in discussion as no single paper can possibly do. The difficulty in the work of such a committee, however, is little realized by those who have not undertaken it. The Committee on Rails took four years in which to formulate a report on simple shapes, with few items in theory or practice to harmonize. The Committee on Tests of Material worked five years under similar conditions, and, after repeated unsuccessful attempts to have meetings, formulated a brief report by correspondence. It was with appreciation of the difficulties of such work that the present specifications were offered for discussion, and it is gratifying to note that those taking part have done so in the same spirit with which they were offered, and have not confined themselves to intangible generalities, but have made most valuable suggestions and modifications.

As has already been noted, railroad specifications should be written from the standpoint of the railroad company, and in so doing every economy will be considered, but it is the economy of durability rather than that of inferior workmanship.

It is to be regretted that anyone should hesitate to discuss matters of opinion. Specifications can hardly be written without them, and an opinion which will not bear discussion and possible modification is of doubtful value.

A careful reading of the paper will probably make plain any apparent ambiguity in regard to compression in members subject to alternate strains. Mr. Greiner did not state whether the required section found by addition would be the necessary net or gross area.

The opinion expressed against chemical analysis is not in conformity with the present practice. No short-time physical tests can be made which will detect micro-flaws due to high sulphur. How does Mr. Greiner provide for this, or does he consider it the invention of "theorists"?

The opinion in favor of drifting tests is directly contrary to that held by the late James G. Dagron, M. Am. Soc. C. E., who, for many years, was Chief Inspector of the Baltimore and Ohio Railroad, and later Engineer of Bridges of that road.

Mr. Seaman. In Mr. Greiner's statement that the specifications do not provide against bad details, he has probably overlooked the clause which requires all working drawings to be submitted for the approval of the railroad company.

The series of quotations cited by Mr. Wilson in corroboration of the 2 to 1 ratio is most timely and pertinent, and is the strongest endorsement possible for this ratio. When an examination of the fatigue experiments indicates a corresponding ratio, it demonstrates that this item may be included in the same proportion. It is certainly the simplest ratio conceivable and should require the strongest evidence of error to justify its abandonment for any other method of proportioning.

When Mr. Cooper questions fatigue, but adopts the 2 to 1 ratio to cover "contingencies unknown"; and others, realizing the existence of fatigue, note that it should be provided for in the same ratio, the margin of difference is very slight.

The increased impact on short span has long been recognized, and recent experiments give more definite light upon this subject, though much yet remains to be learned.

The tables published by Mr. Wilson are interesting records of the method originally adopted by him in the use of the Launhardt formula. It is doubtful if there is any column table adopted since this, which is as quickly applied to new work.

It is unfortunate that a difference should have arisen as to the equivalent of the German centner per square inch, because it does not affect the comparative results, and tends to confuse a subject already somewhat abstruse.

By *Engineering*, 1 centner per square inch (German) = 107 lbs. per square inch (English).

By Professor Marburg, 1 centner per square inch (German) = 104 lbs. per square inch (English).

It appears that the German inch is itself variable in the different German kingdoms, and that Professor Marburg, after careful investigation, has probably reached the correct value, but to avoid confusion in the discussion the author has for the present paper adhered to the value given by *Engineering*.

The criticism that the results on wrought iron were obtained by strains of different kind is correct, as shown on page 142, but the statement that the formula for transverse strain does not apply to strains beyond the elastic limit, is irrelevant, since the formula was only used where the strains were within that assumed limit. The value of 300 centners (32 100 lbs.) which Professor Marburg uses, does not conform to the published tests; for though 32 100 lbs. (300 centners) was applied indefinitely, so also was 34 240 lbs. (320 centners). It is true that the tension tests of 34 240 lbs. resulted

in rupture, but this was only after 10 141 645 repetitions, which would at least indicate that it was very close to the correct value. The value of 47 080 lbs., 25 680 lbs. was carried only to 4 000 000 repetitions. It is not clear, therefore, why these values are not truly comparative. Mr. Seaman.

Professor Marburg has introduced Groups III and IV, but states that data were not available by which he could plot Group III in Fig. 5. It was for a similar reason that the author excluded these groups altogether. They are not sufficiently complete to prove that they are not abnormal, and are of little value in comparison with Group II. The author's purpose in the 2 to 1 ratio was more especially in comparison with the Launhardt formula. In the application of this ratio to bridge construction the element of fatigue becomes only a partial factor.

It is not clear why the fourth and fifth items of Group I, Table No. 5, are added, since they are not included in Wöhler's duration tests. The same criticism applies to the fifth and sixth items of Group II.

Fig. 5 shows a noticeable uniformity, even in the discrepancy of the tests from the line $c\,b$ and would approximate a line running from $5\frac{1}{2}$ at c to 11 at b. It is to be regretted, however, that the paper should have been misconstrued as advocating a precise interpretation, even of the 2 to 1 ratio, as it was this very precision which the author wished to avoid, and believed he had disproved. It was expressly stated that the experiments were "too incomplete to be considered as a demonstration of an exact law," and the "deductions were by no means conclusive." but "the ratio receives much more confirmation than does the Launhardt formula." This last statement remains untouched.

Professor Marburg evidently regards the theory of work, as one to be measured by force × distance, since he considers the records of deformation as necessary to that theory. The word "work," however, was used in the sense of an exertion of strength, whether it be physical work, brain work, or other form of energy.

There appears to be some difference between the interpretation of Bauschinger's results by Professor Marburg and by the author. It is unfortunate that no authorized translation has yet been made, but Professor Marburg has evidently failed to grasp the difference between the "elastic limit" as formerly understood, and that which has been modified to meet Bauschinger's experiments. According to the author's memoranda these experiments showed that the old definition of elastic limit, based upon the absence of permanent elongation must be abandoned from the beginning, since the permanent elongation is never entirely eliminated after a strain is once applied. If the term "elastic limit," or better, "elastic strength," is retained, it must be with Bauschinger's new definition, as limit of uniform elongation.

Mr. Seaman.

Bauschinger also found a second limit which he termed the "limit of strength" (*streck grenze*). Within this second limit the elongation does not increase under continued action of a static load; nor does it diminish after the load is relieved. Beyond this second limit the elongations increase under static load, during several hours or possibly days; and after the load is removed, the elongations decrease.

Bauschinger made some very interesting deductions, too profuse for insertion here, and some of which may perhaps be questioned in view of experiments made by others, both before and since that time, but they throw new light upon Wöhler's tests and their practical application. Among the most pertinent of his deductions are those which describe the almost infinite number of "elastic limits" which may develop under various conditions, and also that which explains that when the "elastic limit" is exceeded by strains in one direction, it is at once completely destroyed for strains in the opposite direction; thus possibly explaining the fact described by Mr. Morison, that columns subject to compression apparently fail by buckling in tension. Bauschinger's statement that when a piece has been subject to strains repeated many times, the resistance to rupture under static load is not diminished by them, but rather is increased, is also of the greatest importance, in view of the effort made by some, to limit the dead strain to values "well within the elastic limit."

If the chief interest in Wöhler's experiments were found in the fact that they proved "(*a*) that innumerably repeated stresses of like character, not exceeding the primitive elastic limit, cannot produce rupture, and (*b*) that the safe limit for equal stresses, opposite in character, is far below the primitive elastic limit," their value would be limited indeed. These facts have long been recognized and provided for. The real value of Wöhler's experiments and of those of Bauschinger, which followed, lies in the fact that they show approximately what may be expected beyond the "elastic strength," and thus make possible an additional factor of safety and economy in bridge design.

The author does not share the regret, kindly expressed by several, that the prevailing tendency of discussion is to criticise. Such a discussion by men who are recognized authorities upon the subject of which they write is probably the most severe test to which a proposition can be subjected, and is a wholesome restraint upon radical and purposeless innovations.

The specifications in Appendix B have been revised by the light of the discussions received.

APPENDIX B.

SPECIFICATIONS FOR STEEL RAILROAD BRIDGES (REVISED.)

Proposals and Drawings.

Preliminary plans.

1. Each bidder shall submit with his proposal complete strain sheets, showing loads assumed in calculations and the resulting strains, and the sections used. The strains from each kind of load shall be shown separately, and the dead load assumed shall not be less than that of the finished structure. At the same time, each bidder shall submit general plans of the structure, and such details as will show clearly the construction of all members and connections. In case of draw bridges, all machinery shall be similarly shown.

2. All drawings furnished by the railroad company, whether of general location or of details of construction, shall be strictly followed.

Drawings approved.

3. Upon the award of the contract and before work is commenced, a complete set of working drawings, in duplicate, including strain sheets and general drawings previously mentioned, shall be submitted to the railroad company for approval. All material ordered or work done before the drawings are approved shall be at the risk of the contractor.

Uniform sheets.

4. All drawings shall be made on the dull side of tracing cloth, and of a uniform size of 24 x 36 ins. After the work is completed, these drawings, in good condition, shall become the property of the railroad company for file. The contractor may retain such prints or copies of them as he may desire for record.

General Provisions.

Type of truss.

5. The structure shall be wholly of rolled steel and wrought iron. A type of truss shall be used in which the strains may be readily calculated and which subjects no pin-connected member to alternate strains. Continuous girders will be allowed only in case of upper chords carrying floors, and in special cases of draw bridges.

6. Double-track, through-truss bridges shall have only two trusses, and four-track bridges only three trusses, unless otherwise specified. In the case of plate girders, special provisions shall be made for spreading the tracks when necessary.

7. All through pin-connected bridges shall have floor beams connected by a web, with both segments of the posts. Hip verticals shall be made rigid. All spans shall have rigid end panels of lower chord when required.

8. When in deck-plate girders the variation in thickness of upper flange plates exceeds 1 in., a separate floor system of stringers shall be provided to carry the ties.

Clearance. 9. The distance from center to center of double track is 13 ft.*

10. All through-bridges, on tangents, shall have a clear opening as shown on the clearance diagram, Fig. 7, or as may be required by local State laws. The width shall be proportionately increased for two or more tracks. On curves, the height and width shall be increased as required by curvature and elevation of rail.

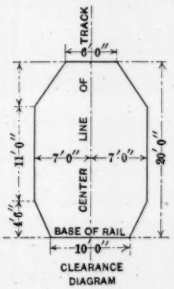


FIG. 7.

Floor system. 11. All bridges shall be provided with a steel floor system. Stringers and deck-plate girders shall be spaced a distance apart between centers equal to their depth, with a minimum limit of 6 ft. 6 ins., unless otherwise required.

Wooden floor. 12. The wooden floor of ties and guard rails will be furnished and put on by the railroad company, unless otherwise specially provided by contract. The cross-ties shall be of long-leafed yellow pine, 8 ins. square and 9 ft. long, spaced 6 ins. apart in the clear and notched $\frac{1}{2}$ in. over the stringer. In cases where the stringers are spaced more than 6 ft. 6 ins. apart, the depth of the tie shall be increased so that the strain on the outer fiber does not exceed 1 000 lbs. per square inch, considering the weight of a single driver as being carried by two ties.

13. Guard rails of long-leafed yellow pine, 6 x 8 ins. shall be placed 3 ft. 9 ins. in the clear, from the center of the track. They shall be notched $\frac{1}{2}$ in. over the cross-ties and spliced with a horizontal half and half joint, 6 ins. long over a tie. They shall be bolted to the cross-tie at the splice, to the cross-tie next adjacent to the splice, and to every fourth tie between, by $\frac{3}{4}$-in. bolts. To all other ties they shall be fastened by $\frac{3}{4}$-in. square spikes. Inside rail-guards shall be placed at a distance of 7 ins. in the clear inside the track rail, and shall extend back over the approach to a point not less than 50 ft. from the back wall.

LOADS.

14. The structure shall be designed to resist the strains from the following loads:

Dead load. 15. The dead load shall consist of the entire weight of the structure, properly distributed at the various panel points.

* This distance may vary according to the practice of different roads.

16. The weight of rails, guard rails, splices and bolts shall be estimated at 175 lbs. per lineal foot of track: ties of standard dimensions at 225 lbs. per lineal foot of track, and special ties at 4½ lbs. per foot B. M.

17. The live load shall be the moving load, with impact. The moving load shall consist of two typical consolidated engines, followed by a train, and distributed as shown in the diagram, Fig. 8. Live load.

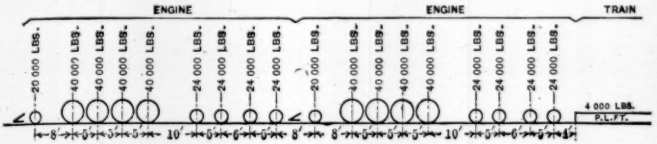


FIG. 8.

18. There shall be as many trains as tracks, each placed in such position as to produce the maximum strains in the structure.

19. The impact will be the increase of live strain, due to the sudden application of the moving load, and shall be provided for according to the following table by interpolation: Impact.

L (ft.)	0	10	20	30	40	50	60	70	80	100
I (%)	50	28	20	14	10	7	4	2	1	0

Where L = Distance in feet through which the moving load must pass to produce the given strain.

I = Percentage of increase of moving load.

20. When the bridge is on a curve, the lateral bracing and vertical trusses shall be proportioned to resist the centrifugal force due to as many trains as there are tracks, moving in the same direction, at the rate of 60 ft. per second. Centrifugal force.

21. Provision shall be made for the sudden starting or stopping of trains, estimating the coefficient of sliding friction at 20 per cent. Sliding friction.

22. Provision shall be made for wind pressure, acting in either direction horizontally of 40 lbs. per square foot of the surface of all trusses and the floor, as seen in elevation, considering the ties solid area; in addition to a train 10 ft. high, beginning 2 ft. 6 ins. above base of rail, and moving across the bridge. In the case of plate girders, only one girder need be considered. The wind forces shall be properly distributed between the upper and lower chords. Wind pressure.

23. Provision shall be made for a variation of temperature of 150° Fahrenheit. Expansion.

ALLOWABLE STRAINS.

24. The live strain will be the total variation in strain produced by the live load. The dead strain will be the minimum strain to which a member is subjected. Live and dead strains.

25. The allowable live strain per square inch shall not exceed one-half of that permissible for dead strain.

26. The allowable dead strain per square inch for rolled steel shall not exceed 18 000 lbs.

27. When wrought iron is used, the allowable dead strain per square inch shall not exceed 16 000 lbs.

Columns. 28. For columns subject to direct compression only, the allowable working strain of 18 000 lbs. per square inch shall be reduced, in proportion to the ratio of length to least radius of gyration, by the following formula:

$$p = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{18\,000\ r^2}}$$

Where p = allowable dead strain per square inch, in pounds,
l = length of column, in inches,
r = least radius of gyration of cross-section, in inches.

29. For columns subject to alternate strains of tension and compression, the compressive strain shall be increased to provide for the increase of strain due to flexure in proportion to the ratio of the length to the least radius of gyration by the formula:

$$P = p\left(1 + \frac{l^2}{18\,000\ r^2}\right)$$

Where p = direct compression in member,
P = increased strain due to length of column,
l and r = length and least radius of gyration of cross-section, in inches.

30. The increased compressive strain per square inch (gross) thus found shall be added to the tensile strain per square inch (net), and the resulting total unit strain shall not exceed that specified for live strain.

Compression flanges. 31. In the case of compression flanges of beams and girders, the allowable working strain per square inch of such flanges for dead strain shall be computed by the formula:

$$C = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{5\,000\ w^2}}$$

Where C = allowable compressive strain per square inch,
l = unsupported length of compressed flange, in inches,
w = width of flange, in inches.

Strains on pins, rivets, etc. 32. The shearing on pins, rivets, and bolts shall not exceed for dead strain 13 500 lbs. per square inch of cross-section. Where tension on rivets is unavoidable, it shall not exceed one-half the limit allowed for direct shear. When a force is oblique, the components of

direct tension and of direct shear shall be considered separately and the results combined.

33. The bending strain in the outer fiber and the bearing pressure (diameter × thickness) on pins, rivets and bolts, shall not exceed for dead strain 27 000 lbs. per square inch.

34. In cases of field rivets, driven by hand, an excess of 25% shall be allowed.

35. Wind pressure and sliding friction, either separately or combined, shall not produce greater strain per square inch than that allowed for dead strain. Centrifugal force shall be considered as live strain. Lateral strains.

36. The pressure, in pounds per lineal inch of roller, shall not exceed $1\,200\sqrt{d}$ for dead load (d = diameter, in inches). Rollers.

37. Bed plates on masonry shall be so proportioned that the greatest pressure on the masonry does not exceed 300 lbs. per square inch. Bed plates.

Details of Design.

38. The assumed spans for calculation shall be as follows: Span for calculation.

For pin-connected trusses—distance between centers of end pins.
" riveted girders " " " bearing.
" cross-girders " " " trusses.
" track stringers " " " of cross-girders.
" cross-ties " " " of track stringers.

39. The assumed depth for calculation shall be: Depth for calculation.

For pin-connected trusses—distance between centers of chord pins.
" riveted lattice girders " " " " gravity of flanges (not to exceed the distance out to out of angles).
" plate girders—distance out to out of angles.

40. The whole of the wind force due to the train and floor, and one-half of the truss, shall be considered as acting on the lateral system of the loaded chord, and that due to one-half the truss only, on the lateral system of the unloaded chord. Lateral bracing.

41. In the case of deck bridges and very heavy curves, some of the centrifugal force may be transferred to the lower lateral system, in which case the truss shall be duly strengthened. The end portal bracing in through bridges must be of sufficient strength to transfer the accumulated wind strains from the upper lateral system to the end posts, and the end sway-bracing in deck bridges shall carry the whole of the accumulated wind and centrifugal forces from the loaded chord to the abutment.

42. Each main panel of deck bridges shall be provided with intermediate sway-bracing of a sufficient section to carry one-half the maximum increment due to wind on train, and to centrifugal force. Through bridges shall be provided with post brackets at the intermediate panel

points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure, or, when the height of top chord exceeds 25 ft. above base of rail, an overhead system of sway-bracing shall be used.

43. Lateral and sway-bracing shall be designed to receive an initial tensile strain in the diagonals of 10 000 lbs. or shall be capable of sustaining a compressive strain of one-half that amount.

44. Tension at the windward column of trestle piers shall be avoided if possible, and in any case approved anchor bolts well secured to the masonry shall be used.

45. The struts shall be proportioned to withstand their component of strain. No reduction shall be made from chord section on account of the material in the lateral system.

Design of details.

46. All parts shall be so designed that the strains coming upon them may be definitely calculated. The center line of resistance of a member will be along its neutral axis, and connections shall be so designed as to avoid bending, twisting or unequal tearing of the member or its details. The line of strain shall pass centrally through any cluster of rivets which resist it, and where angles or plates are otherwise than so connected, proper provision shall be made for the moments and secondary strains produced. Details shall be so designed as to give free access for inspection and painting, and water pockets shall be avoided. In every case the connection of details shall be of greater strength than the member itself.

47. All members which are subject to direct strains, in addition to bending moments, shall be so proportioned that the algebraic sum of the strains coming upon them shall not exceed the specified allowable strain, properly reduced in case of columns. In continuous upper chords of deck bridges, carrying the floor, the strain due to the live load shall be computed from a bending moment equal to $\frac{3}{4}$ of the maximum moment produced by the engine on a span equal to a panel length considered as a simple beam.

48. The strain on the outer fiber of solid shapes shall be computed from the moment of inertia of the section.

Plate girders.

49. No allowance shall be made for the web in calculating the flange section of plate girders. Girders formed of web plates and angles alone, having no upper flange plate proper, will be allowed only when ties rest upon the upper flange. At least one upper flange plate, when used, shall extend from end to end of the girder, and any additional plates used to make up the flange section shall be made of such lengths as to allow at least two rows of rivets of the regular pitch being placed at each end of the plate beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of the plates to transmit their value before the theoretical point of the next outside plate is reached. Where the flange plates vary in thickness, they shall decrease

outward from the flange angles. All plate girders shall have end and corner cover plates. The total thickness of plates and angles shall not exceed five times the diameter of the rivet used.

50. All flange plates, subject to either tension or compression, spliced in the length of the girder, shall be covered with an extra amount of material equal in section to the material spliced, with sufficient rivets on either side to transmit the strains from the parts cut. Flange angles shall be spliced with angle covers.

51. In calculating the shearing or bearing strain in web rivets of plate girders, the whole of the shear acting on the side of the panel next to the abutment shall be considered as being transferred into the flange angles at a distance equal to the depth of the girders.

52. The webs of plate girders shall be spliced, wherever cut, by a plate on each side of the web capable of transmitting the full shearing strain through splice rivets.

53. When the thickness of the web plate is less than $\frac{1}{50}$ of the unsupported distance between flange angles, stiffeners shall be riveted on both sides of the web, with a close bearing against the upper and lower flanges and calculated as columns by the compression flange formula for the whole shear at the several points where they are placed. The distance, center to center, of stiffeners, shall generally not exceed the depth of the full web plate, but shall not be less than 4 ft. Web plates generally shall have stiffeners at all splices, at points of concentrated loading, and at each end of bearing plates.

Net sections.

54. Net sections shall be used in all cases in calculating tension members, and in deducting rivet holes they shall be taken as $\frac{1}{8}$ in. wider than nominal diameter of rivet. In calculating the net sections, having rivets staggered, all rows shall be deducted, unless so arranged that the net section along a zigzag line, taking all distances in the diagonal direction at only three-fourths their value, exceeds the corresponding net section directly across the plate.

Rivet spacing.

55. Rivets shall not be spaced closer than three diameters, center to center, nor further apart, in the direction of the strain, than sixteen times the thickness of the thinnest external plate connected, and not more than thirty times that thickness at right angles to the line of strain.

56. Rivets shall not be spaced closer to the side of the plates than $1\frac{1}{4}$ diameters to the center of the rivet, nor further from the side than eight times the thickness of plate. In no case shall the pitch of rivets exceed 6 ins.

57. Field rivets shall be reduced to a minimum.

Chord splices.

58. Built chords shall be thoroughly spliced with rivets and additional section sufficient to transmit the entire strain; no allowance shall be made for abutting surfaces, except in work exceeding $\frac{3}{4}$ in. in thickness.

Pin plates. 59. When necessary to obtain sufficient bearing surface at pin-holes, re-enforcing plates shall be added. These plates shall distribute the bearing strains from the pins to the member to which they are connected.

Compression members and latticing. 60. All segments of members in compression, connected by latticing only, shall have batten plates at each end, the thickness of which shall not be less than $\frac{1}{40}$ of the distance between rivets connecting them to the compressed member. In no case shall the length of the batten plate be less than $1\frac{1}{4}$ times the width of the member.

61. The distance between connections of latticing shall be such that the individual members composing the column, considered with hinged ends and a length equal to the distance between these connections, shall be stronger than the column as a whole, and in no case shall this distance exceed eight times the least width of these members. Where the ends of the compression member are forked to connect the pins, the strength of each leg shall be at least equal to the entire strength of the column, and the re-enforcing plates shall extend not less than 6 ins. beyond the edge of the batten plates.

62. Single lattice bars shall have a thickness of not less than $\frac{1}{60}$, and double lattice bars not less than $\frac{1}{50}$ of the distance between the rivets connecting them to the compressed member, and their widths shall be:

For—

33 ins. and over, web,	4-in. angles	($2\frac{7}{8}$-in. rivets),	3 x 2 x $\frac{3}{8}$-in. angles.
29 ins. to 32 ins.,	"	"	$2\frac{1}{2}$ x $2\frac{5}{16}$-in. "
20 ins. to 28 ins.,	"	"	4-in. straps.
15 ins. to 19 ins.,	"	($1\frac{7}{8}$-in. rivets),	4-in. "
12-in. channels, or 3-in. angles,		"	$2\frac{3}{4}$-in. "
9-in. "	" $2\frac{1}{2}$-in. "	($1\frac{3}{4}$-in. rivets),	$2\frac{1}{2}$-in. "
8-in. "	" 2-in. "	"	2-in. "

63. Single lattice bars shall generally be inclined at an angle of 60° to the axis of the member, and double lattice bars at an angle of 45°, with a rivet at their intersection.

Expansion rollers. 64. All bridges over 80 ft. long shall be provided at one end with turned friction rollers, not less than 3 ins. in diameter, between two planed surfaces. For spans of 80 ft. or less, planed surfaces shall be used without rollers. The nest of rollers shall be easily cleaned from the accumulation of dust and cinders.

Bearings anchored. 65. Trusses shall be secured against side motion on bearing plates and rollers. The bolster blocks shall be joined to the truss, and the bearing plates shall be secured to the underlying supports by bolts or dowels.

Eye-bar packing. 66. Eye-bars shall be so packed as to produce the least bending moment on the pin, and shall not be packed out of line with the axis of the member more than $\frac{1}{8}$ in to 1 ft.

67. No iron less than $\frac{5}{8}$ in. thick shall be used, except for packing or other idle material. No counter rod shall have less than $1\frac{1}{2}$ sq. ins. of sectional area. Minimum section.

68. The camber shall be such that under maximum load the bridge will not deflect below a horizontal position. Camber.

Quality of Material.

69. *Rolled Steel.*—Rolled steel shall be made by the open-hearth process, and shall contain not more than 0.04% phosphorus, 0.04% sulphur, nor 0.45% manganese. The steel shall be finished straight and smooth, and shall be a perfect product; the slightest flaw will be sufficient cause for rejection at any time during the progress of the work. Chemical analysis and finish.

70. The tensile strength, yield point, and ductility of the material shall be determined from a standard test piece of not more than 2 ins. in width, nor less than $\frac{1}{4}$ sq. in. in sectional area, cut from a full-sized bar, and with sides turned or planed parallel, so as to give a uniform minimum section for a length of at least 12 ins. Standard test piece.

71. Whenever practicable, the two sides of the test piece shall be left as they come from the rolls, but the finish on opposite sides shall be alike in this respect.

72. In determining the ductility, the elongation shall be measured after breaking, on an original length of 8 ins., in which length shall occur the curve of reduction each side of the point of fracture.

73. The yield point shall be that strain beyond which the elongation ceases to be proportional to the weight imposed, and may be indicated by "drop of beam." It shall in no case be less than 55% of the maximum strain sustained by the test piece. The speed of testing shall be governed by the inspector. Yield point.

74. All rolled steel, except rivet steel, shall show by the standard test piece a maximum strength per square inch of 56 000 lbs. $\pm$ 4 000 lbs., with an elongation of 26% in 8 ins. Rivet steel, when tested in specimens of full size of rivet rod, shall show an ultimate strain per square inch of 52 000 lbs. $\pm$ 4 000 lbs., with an elongation of 30% in 8 ins. Maximum strength and elongation.

75. Each melt of finished material shall receive two tension tests—one cut from each extreme variation in thickness of metal rolled. When both tests comply with the specifications, all intermediate thicknesses will be accepted; otherwise only such thicknesses of metal will be accepted as show satisfactory tests.

76. Each finished piece of steel shall be marked with the melt number. Material marked.

77. *Wrought Iron.*—All wrought iron shall be tough, ductile, fibrous and uniform in quality. It shall be thoroughly welded in rolling, and finished straight and smooth. It shall be free from flaws, blisters, Finish.

cinder-spots, cracks and imperfect edges. Scrap steel shall not be used in its manufacture.

78. The methods specified for testing rolled steel shall apply generally to wrought iron.

Maximum strength and elongation.

79. All iron shall show by the standard test piece a maximum strength of not less than 50 000 lbs. per square inch and an elongation of 20 per cent.

Yield point.

80. The yield point, as shown by the standard test piece, shall in no case be less than 26 000 lbs. per square inch.

Bending tests.

81. All iron when cut into testing strips $1\frac{1}{2}$ ins. in width and with corners rounded to $\frac{1}{8}$ in. radius, must be capable of resisting, without signs of fracture, bending cold 90° with the inner radius not to exceed three times the thickness of the test piece.

82. All iron which is to be bent in manufacture shall, in addition to the above requirements, be capable of bending sharply to a right angle at a working heat without any signs of fracture.

83. *Cast Steel.*—Steel castings shall be true to pattern and free from injurious blow holes or other imperfections. Coupon specimens, turned to $\frac{3}{4}$ in. diameter, shall show an ultimate strength per square inch of 65 000 lbs., a yield point of 35 000 lbs., and an elongation of 15% in 2 ins.

84. *Cast Iron.*—All cast iron shall be tough and sound, free from blow-holes, cold-shuts or other injurious imperfections. When broken, the fracture shall indicate a good quality of gray iron.

85. Sample test specimens, 27 ins. long, 2 ins. × 1 in. in cross-section, cast under the same circumstances as those which attend the casting of the full-sized piece, shall sustain at the center, when resting flatwise upon two dull knife edges, spaced 24 ins. apart, a load of 2 000 lbs.; the load to be sustained two minutes, and show a deflection of not less than $\frac{1}{4}$ in. before fracture.

Workmanship.

Finish.

86. All workmanship shall be first class. All parts exposed to view shall be neatly finished. All nuts shall be hexagonal.

Punching.

87. In punching, the diameter of the punch shall not exceed by more than $\frac{1}{16}$ in. the diameter of the rivet to be used, and the diameter of the die shall be as small as may be required to punch a clean hole.

88. The holes shall be so carefully spaced and punched that, upon assembling, no variation from a truly opposite position of more than $\frac{1}{16}$ in. will occur. All holes shall be reamed to an exact match before the rivet is driven.

Rivets.

89. Rivets when driven, shall completely fill the holes and shall be machine driven whenever possible. They shall have full, con-

centric, hemispherical heads of a depth at circumference of shank of not less than one-half the diameter of rivet, and with full bearing on the plates; or they shall be countersunk when so required. Rivet heads shall not be flattened to less than half the diameter of the rivet unless countersunk.

Turned bolts.

90. Generally, the use of bolts instead of rivets will not be permitted, but when used in special cases the holes shall be reamed parallel, and the bolts turned to a driving fit.

Template reaming.

91. All holes for field rivets, excepting those in connections of lateral and sway bracing, shall be accurately drilled to an iron template, or reamed while the connecting parts are temporarily assembled in the shop. In the case of splices of upper chords, or other compression members, they shall be brought to forcible contact by the use of turnbuckles, and after reaming shall have match marks put on the pieces so that they may be brought to proper position in the field, before riveting.

Members true.

92. Finished members shall be true and free from kinks, twists and open joints. Ends of floorbeams shall be finished square and true.

Tensile rivets.

93. Where rivets may take tension, the rivet holes shall have edges rounded to $\frac{1}{8}$ in. radius.

Screw threads.

94. Rods and bars which are to receive a thread shall be properlyupset. Where threads are cut on steel, they shall be properly filleted.

Adjustable members.

95. All members requiring adjustment shall be provided with sleeve nuts and check nuts. Open turnbuckles will not be allowed. The ends connected shall be distinctly punch-marked, at a distance of 12 ins. from the screw ends, so that these ends may be accurately located inside the nut.

Eye bar heads.

96. Heads of eye-bars shall be of sufficient section to break the bar in every case. Bars shall be full size at the neck; no patching will be allowed.

97. Welds in the body of the bars or rods will not be allowed.

98. Pins shall be turned, perfectly finished, and straight.

Boring.

99. All members having bearing on pins shall be carefully bored at right angles to the axis, unless otherwise shown in the drawings. No variation will be allowed between diameter of pin and pin hole of more than $\frac{1}{32}$ in. For pin holes, in pieces which are not adjustable for lengths, no variation of more than $\frac{1}{64}$ in. in length between centers of pin holes will be allowed.

100. Eye-bars shall be perfectly straight before boring; the holes shall be in the center of the heads and on the center line of the bar. Bars which belong to the same member shall be bored at the same temperature and in one operation. They shall be marked for erection, so that they may be used in the same member.

Abutting surfaces. 101. All abutting surfaces, except flanges of plate girders, shall be neatly planed or turned perpendicular to the direction of the strain, so as to insure even bearings.

Stiffeners. 102. Stiffeners of plate girders shall be milled to fit tightly against the flange angles, and shall be packed straight. The packing shall fit close to the flange angles, leaving no open space.

Rollers turned. 103. Rollers shall be turned and roller beds and bed plates planed; the bottom of shoes shall be planed exactly parallel to the center line of pin, unless otherwise shown in the drawings.

Thickening washers. 104. Thickening washers shall be used whenever required to pack the pin joints tight.

Pilot nuts. 105. Pilot nuts shall be furnished for each size of pin to preserve the thread of the pin, and to facilitate erection.

Painting.

106. All iron shall be scraped free from scale, and receive one coat of pure kettle-boiled linseed oil before leaving the shops, and one coat of approved paint after erection. All surfaces which come in contact, or are enclosed, shall receive one coat of approved paint before being assembled.

107. All turned or faced surfaces shall receive a coat of white lead and tallow before leaving the shops.

Inspection.

108. Free access and information shall be given by the contractor for a thorough inspection of material and workmanship.

109. The inspector will make detailed reports of his inspectin to the engineer and may notify the contractor of any defects in material and workmanship, but all acceptances made by him shall be considered as temporary, and his inspection shall in no way relieve the contractor of full responsibility for the character and accuracy of the work until its completion and final acceptance by the engineer.

110. The contractor shall furnish without extra charge such standard test pieces as may be necessary to determine the uniform quality of the material, and also the use of a reliable testing machine with necessary labor for testing.

111. Full-sized members may be tested to destruction by the engineer. They will be paid for at cost, less their scrap value, if they fulfill the requirements, but when the requirements are not complied with, the tests shall be at the expense of the contractor. Such rejections of the finished material as he may consider warranted by the results of these tests will be made by the engineer.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 846.

THE COAL HOISTS OF THE CALUMET AND HECLA MINING COMPANY.

By JULIUS KAHN, Jun. Am. Soc. C. E.

PRESENTED JANUARY 4TH, 1899.

WITH DISCUSSION.

The fact that large quantities of coal are consumed daily by manufacturing, mining, and commercial industries has necessitated the introduction of machinery especially adapted for handling this material. The loading of coal from cars into freight vessels, or from vessels into cars, has brought about the use of certain classes of this machinery—the unloading into coal storage sheds, and again the delivery from the sheds directly to the boiler stokers, has created the need of others. Hoisting and conveying machines to do this work are so varied in their construction to suit different conditions, that machinery of this class can in itself form a distinct branch of engineering.

The author has been unable to find in the *Transactions* any paper which has dwelt on this subject, and it has seemed to him that the description of one very large plant might possibly be of interest, and some of its engineering features worthy of special attention. The plant described was erected for the Calumet and Hecla Mining Com-

pany, during the months of June and July, 1898, at Lake Linden, Mich.

The Calumet and Hecla Mines are located on a peninsula, known as Kewenaw Point, and extending into Lake Superior. The smelting works and coal docks of the company are at Lake Linden, a small town situated on an inlet of Lake Superior; the mines are at Red Jacket and Calumet, Mich., two towns about 6 miles from Lake Linden.

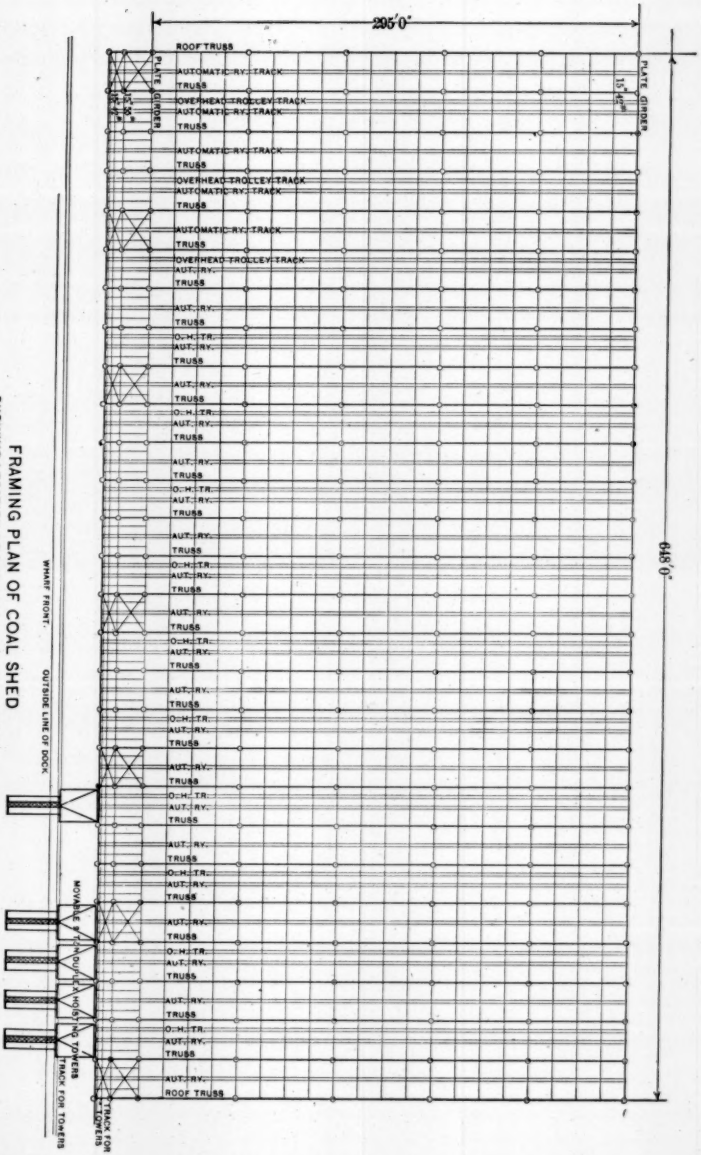
All coal is received from freight vessels at the docks in Lake Linden, and is hoisted there and conveyed into the coal storage sheds. It is carried from Lake Linden to the mines at Calumet by rail; the cars, entering passages provided for them in the sides of the sheds, are filled by special loading devices. The coal-carrying vessels from Buffalo to the upper peninsula have an average capacity of from 1 200 to 1 400 tons. They have usually five hatches, each about 8 ft. by 12 ft., and at 24 ft. centers.

The coal most commonly used is of the grades known as "Youghiogheny" and "Pocahontas," of Pennsylvania, and the "Soldiers' Run," of Virginia, all of which are large-lump, bituminous coals. There is also received a small quantity of anthracite, which serves for the domestic use of the workmen.

In 1894, the coal received by the company amounted to 225 420 tons; in 1895, 175 162 tons; in 1896, 207 000 tons; in 1897, 178 800 tons, and in 1898, 285 000 tons.

It can, of course, be easily appreciated, that even the slightest saving per ton for unloading this enormous quantity of coal from the boats, and conveying it to the coal sheds must amount to a large sum yearly, and, at a low rate of interest, must warrant a very large investment. Coal hoisting with a steam-shovel plant of this character, which takes the material from the hold of the vessel and delivers it into the storage sheds, is done at a possible cost of between 1 and 3 cents per ton. The same work done by stevedores would probably cost from 25 to 33 cents per ton.

The reasons for storing this immense quantity of coal are, primarily, to carry the works through a winter's demand, lake transportation being closed during this season; secondly, the company has found it necessary to make itself absolutely independent of possible accidents in the coal supply, caused by mine strikes and price



FRAMING PLAN OF COAL SHED

SHOWING LOCATION OF AUTOMATIC RAILWAY TRACKS.

FIG. 1.

fluctuations, and, to do this, has erected the steel-framed coal shed shown in Fig. 1, 648 ft. by 295 ft., to be used for storage or as a reservoir to be drawn upon in case any of the before-named causes make the supply less than the demand. The hoisting towers and automatic railways, which are used for unloading the coal from the vessels and carrying it into the sheds, are the subject of this paper.

Fig. 1, Plate II, is a side view of the hoisting towers and the coal shed, taken during erection and before the latter was enclosed. Fig. 2, Plate II, is a front view of the towers.

Fig 1, Plate III, shows the engine floors of the towers before they were enclosed with their housing, all the towers being close together. Fig. 2, Plate III, shows the engine floor of one of the towers, with the engineer at his operating stand.

Fig. 1, Plate IV, shows the wheel-bases of the towers. Fig. 2, Plate IV, is a view of the towers, completed, and several of them engaged in hoisting coal.

Figs. 2 and 3 show elevations of the towers and plans of the floors. A section and general framing plan of the building are shown in Fig. 1.

The towers have been reduced to great simplicity in their general design, and the individual members have been so placed that the theoretical strains are transmitted almost directly to the foundations. The rear posts have been battered to enlarge the base and increase the stability of the structures. It was impossible to increase the width of the towers beyond that shown in the front elevation, the distance between centers of the automatic railways being given as 24 ft., which it was not allowable to exceed. Each tower is mounted on five pairs of heavy wheels, is capable of being moved anywhere along the wharf, so that hoisting can be done from any one of the different hatches in a vessel, and if necessary all towers can be at work at one vessel at the same time. As the building in the rear contains 27 automatic railway tracks, and as any one of the towers is apt to unload on any one of the tracks, the extreme distance moved along the wharf is 648 ft.

The side elevation (Figs. 2 and 3) shows two floors, the upper one being used as the engine floor and the lower one as the car floor. On the upper floor is the receiving hopper and the hoisting engine; the latter is a 12 x 14-in. engine, with double drum and double cylinder, fitted

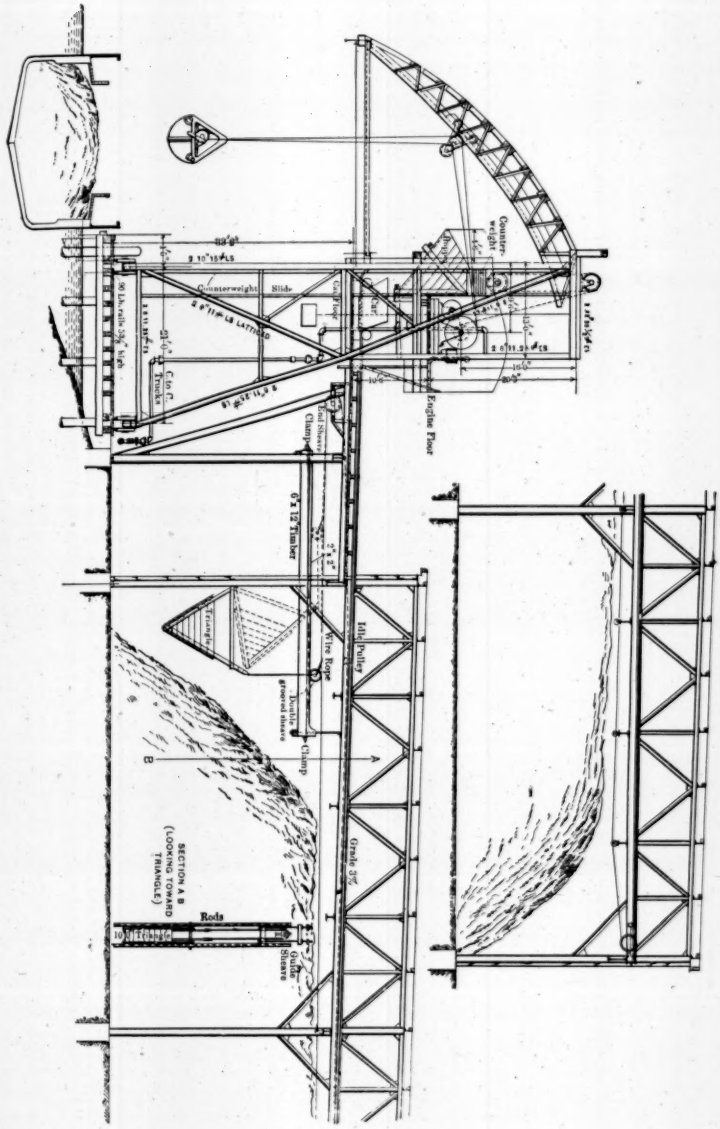

Counter-weight
Counterweight
Slide
Car Floor
Car
Engine Floor
End Sheave
Clamp
6" x 12" Timber
Triangle
Wire Rope
Idle Pulley
Double grooved sheave
Clamp
Grade 3%
SECTION A B
(LOOKING TOWARD TRIANGLE)
Rods
Guide Sheave
C. to C. Trucks
90 Lb. rails 3⅜" high

FIG. 2.

with a steam brake, a steam reversing cylinder, an emergency brake, and a counterweight spool. Its drums are so constructed that the chains, which are laminated, wind upon themselves in a vertical plane; the reasons for this will be evident from the description of the boom, given later. The car floor contains the scale platform and the automatic car, the latter being stationed directly under a trap door in the hopper.

The plans of these floors show them to be heavily trussed in a horizontal plane and the rear elevation shows them to be trussed in a

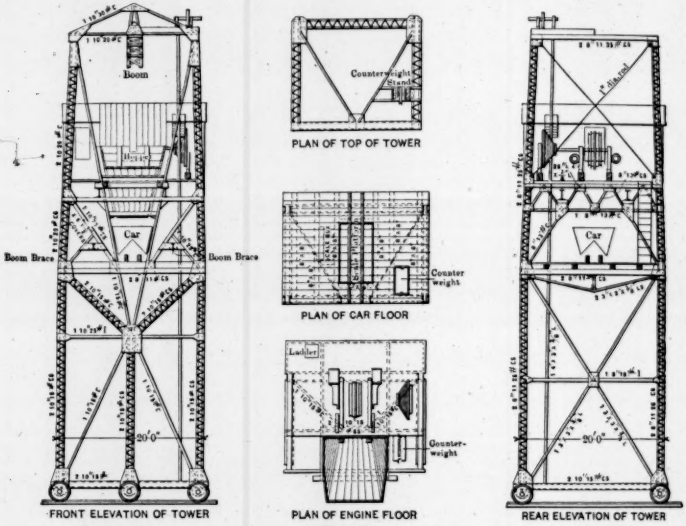


FIG. 3.

vertical plane. The truss under the engine floor takes up the load of the engine and the varying upward and downward pulls of the shovel chains. The horizontal truss under the floor receives the horizontal component of the hoisting chain pull, and carries it into the tower proper.

The boom is a heavily trussed structure, of a peculiar shape, the top chord being the arc of a circle of 64 ft. radius and the bottom chord approximately a parabolic segment. It is made up of two trusses, connected together by angles and plates in the plane of the top chord, and by bracing between the webs.

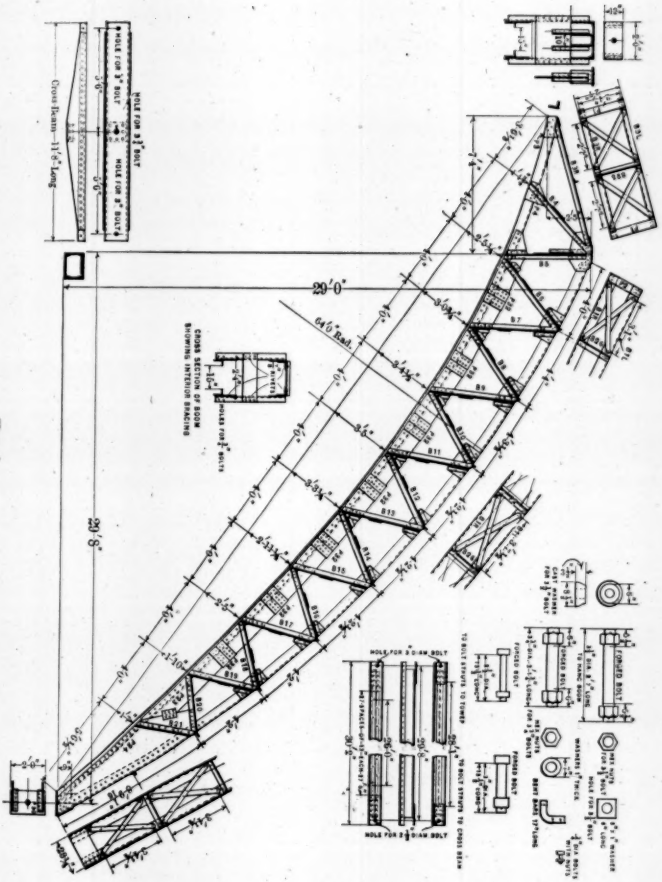


FIG. 4.

The boom truck, containing the sheaves for the shovel chains, moves between the trusses, the flanges of the bottom chord acting as guides and rails. A chock block is used for arresting its downward travel. This is operated by a counterweight of its own, and automatically adjusts itself to different positions along the boom from which it may be desired to do hoisting.

A similar chock block at the upper end of the boom stops the truck directly above the hopper, and holds the same by a projecting hook when the truck is not in use.

At the farther end, the boom is connected to the cross-bar by means of a vertical pin, and transmits its horizontal stress into the same; the cross-bar is again connected by vertical pins with the horizontal braces. At its upper end, the boom is suspended from the top hip truss which is carried on cantilevered beams from the tower. This connection is also made with a large vertical pin. The details of the boom are shown in Fig. 4.

When hoisting is being done, the boom stands out at right angles to the front of the tower, and is held in this position by two guy lines attached to its sides. When not in use, it is swung around horizontally, approaching as nearly as possible to the face of the tower, thus allowing boats with high masts to move along the wharf.

The bottom chord of the boom was so designed that the resultants of the forces, when hoisting is being done at any point, are always normal to the track or bottom chord, a certain factor of safety or extra declivity being allowed for; this factor will be explained somewhat under the heading "Construction of the Boom." The resultant stress on the truck, being always in such a direction as to press it normally against the tracks, it has no tendency to run up the boom while the shovel is suspended from it. When the shovel reaches the truck, it bears on a suspended arm of the latter, and the chain now pulls both truck and shovel toward the tower and over the hopper, where the shovel is allowed to dump itself.

With this scheme of hoisting, where the shovel chains merely pass over the sheaves in the truck, it is of course easily seen that if the curve is not theoretically correct, and possesses a certain factor of safety, there is a tendency to slide the same either up or down the boom as soon as a pull comes on the chain.

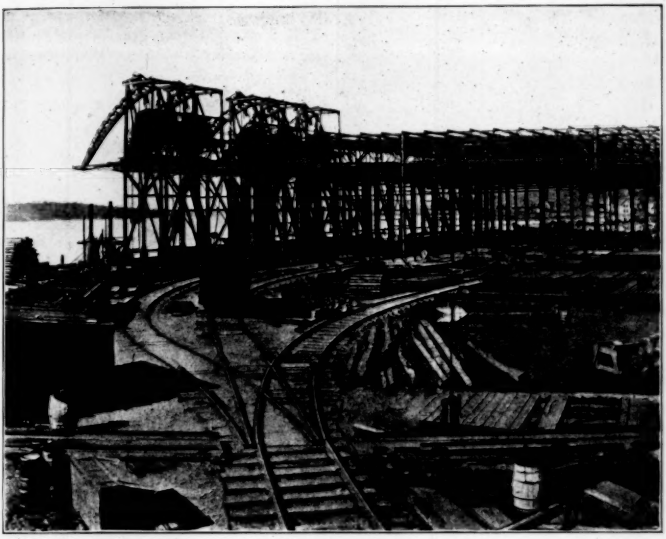


FIG. 1.

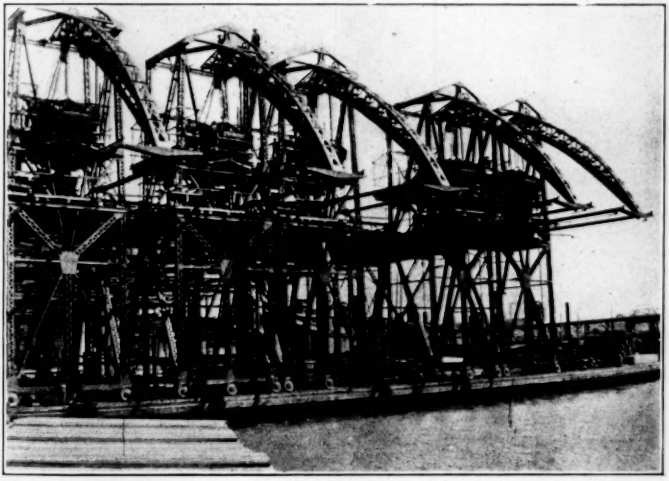


FIG. 2.

Operation of the Hoist.

The engineer is stationed at his operating stand at the front of the tower, as shown in Fig. 2, where he can at all times watch the shovel. He controls the throttle valve with a foot lever; the reversing gear is operated by a steam valve on his right, and the brake band by a steam valve on his left. There is also an emergency hand brake on his right. Two chains are used for operating the shovel, the hoisting and holding chains, both of them being of a laminated type, and made up of ten links.

Beginning with the empty shovel supported over the coal in the boat, the engineer sets his steam brake and supports the shovel on the holding chain. He then moves his reversing lever and allows the hoisting chain to become slack, the shovel opening by its own weight. As soon as the shovel has entirely opened, the engineer frees the brake band, and, with the engine reversed, allows the shovel to fall into the coal pile from a small height. The shovel by its own weight buries itself in the coal. The engineer, having the brake band set so that the holding chain drum is fixed, allows his engine to run forward and raise the hoisting chain. The scoops are thus forced together without exerting any tendency to lift the shovel out of the coal. As soon as the shovel is closed, the engineer frees the brake band of the holding chain drum and raises the shovel by both the holding and hoisting chains, slowing up a little as it approaches the boom, and from this point the shovel moves with the truck to the hopper. The engine is then stopped, and the shovel is suspended with the holding chain. The engine is reversed, which allows the hoisting chain to become slack, and causes the shovel to open and unload itself into the hopper. The shovel is again closed by winding up the hoisting chain; the brake band is released; the engine again reversed, and the shovel is allowed to descend the boom, and into the vessel.

It requires about 50 seconds to make one trip, and 2 tons are hoisted each time.

A man standing on the car floor below operates the trap door in the back of the hopper, and allows the coal to flow into the automatic car, which is stationed on the scale platform. He weighs the coal, and gives the car a slight push to start it down the incline of the automatic railway. It rushes along until at a certain point in its journey

it strikes the cross-bar, a strong oak frame laid across the track and connected by wire ropes with a triangular weight. The car picks up the cross-bar and drags it along, thereby raising the weight. Gradually, as the weight is raised, the energy of motion contained in the loaded car is transformed into the potential energy of the raised triangle. The cross-bar is located at such a distance before the point of dumping, that this energy of the loaded car has been entirely expended in carrying it to the dumping point. The car, therefore, when it arrives there is at a momentary standstill; its sides during the last few feet of motion have engaged the trip block and are caused to fly open. The coal is thus allowed to fall vertically into the shed, without any other component of force than that due to its height of fall, and this insures the least possible breakage of coal.

The car being empty, the raised triangle, possessing the energy of the loaded car, now descends, and forces back the cross-bar, thus giving the car an impulse sufficient to carry it back to the platform in the tower. All this work is done in a very short time, the car running down the grade with a constantly accelerated velocity, and returning with a constantly retarded velocity due to gravity.

Automatic railways of this character have been in operation for many years, especially where coal conveying is done to any extent. The entire arrangement works speedily, costs absolutely nothing for operating expense, is easily constructed, and has no delicate parts to be easily put out of order. A special and interesting feature of its mechanism is the device used to gradually retard the rapid descent of the car, changing its forward motion into a backward motion and giving just enough momentum to carry the car to the starting point.

The working of the triangle offers a very interesting mathematical study, in so far that its action is not the same as that of a falling weight; that its effective pull is of a varying nature, greatest and least at those points where it is most desirable to have it so, and that it receives and imparts great velocity with scarcely any shock or vibration.

The triangular weight is shown in Fig. 5, suspended from a double-grooved sheave which travels back and forth on a 6 x 12-in. timber track. The rope leads from the cross-bar, in one direction over a

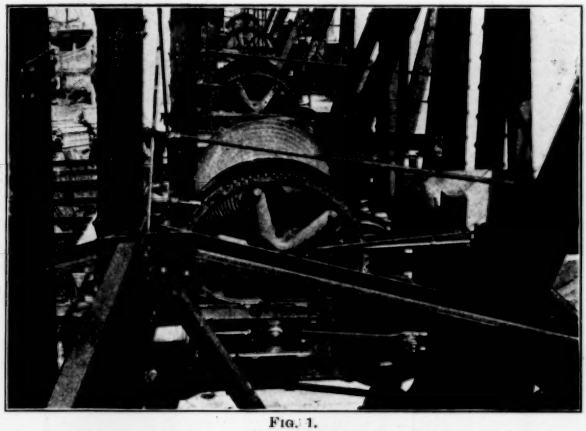


FIG. 1.

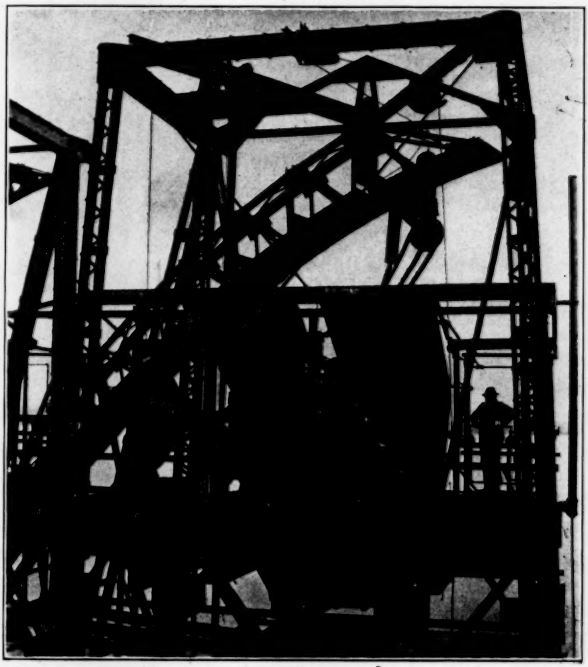


FIG. 2.

sheave at the end of the track, returns below the track to the triangle sheave, leads half-way over this, and is fastened to the end of the track timber pointing toward the rear of the shed. Leading in the other direction from the cross-bar, it passes around the other end sheave, returns below to the triangle sheave, takes a half turn about this and is fastened to the other end of the timber. It will be noticed, that at the instant when the car strikes the cross-bar, at which time its momentum is greatest, the triangle sheave is directly above one end of the triangle, and the force required to move it a very short distance, is theoretically zero if friction were neglected. As the cross-bar travels with the car, the triangle sheave is drawn forward along the timber track and the triangle is raised, its effective weight increas-

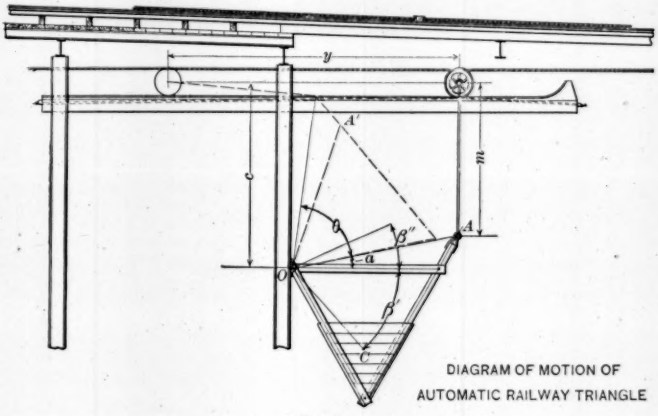


FIG. 5.

ing by a varying quantity. The car is thus gradually brought to a stop, at which instant the coal drops into the shed; then the backward push of the cross-bar on the car takes place, and from the entire effective weight of the triangle the backward push is gradually retarded, until the sheave is again directly over one end of the triangle, when it is reduced to nothing. As the backward push, however, is being retarded, the velocity of the triangle sheave and consequently that of the car is being increased; and at that point where the effective push of the triangle reduces to zero, the sheave velocity is at its maximum.

There is in the triangle and cross-bar a mechanism very nearly identical, in its rebounding effect, with that of the bow and arrow. At

the instant when the bow string is released, the component of string tension pressing on the end of the arrow is very large, and as this component decreases, the transverse velocity of the string increases, until at that point where the string is straight, its effective push is zero. Its velocity, however, is at a maximum, and is infinite as compared with the ends of the bow.

In the short mathematical discussion which follows, the author has deduced formulas to determine the following facts :

First.—Given the weight of the triangle, to derive the angular distance through which the triangle is raised and the distance through which the car drags the cross-bar ; or, given the distance through which the car drags the cross-bar, to determine the weight of the triangle.

Second.—Relative velocity of the rebounding car and falling triangle.

Third.—Real velocity of the rebounding car.

Fourth.—Frictional losses.

Let W = weight of empty car,

W' = weight of coal carried by car,

h = total fall of car from starting point to position of dumping,

δ = angle which rail slope makes with a vertical line,

w = weight of triangle,

w'' = weight of triangle sheave,

$r = O\,A$,

a = distance OC of center of gravity of the triangle from O, the center of rotation,

c = distance from the center of axle to center of triangle sheave,

y = horizontal distance of triangle sheave from its initial position,

θ = angle which any radial line from O makes with its initial position for any position of the triangle,

α = angle OA makes with the horizontal radius when in its initial position.

Total kinetic energy of descended loaded car = work done in raising triangle.

Work of swinging the center of gravity of the triangle through angle $d\,\theta = a\,w\,\cos\,\theta\,d\,\theta$.

First.—Total work of swinging the center of gravity of the triangle = $\int_{\beta'}^{\beta''} a\,w\,\cos\,\theta\,d\,\theta = a\,w\,(\sin\,\beta'' - \sin\,\beta')$, where β' and β'' are the initial

FIG. 1.

FIG. 2.

and final angles made by the radial line through the centroid with the horizontal radial line. But since total work done on the triangle equals total kinetic energy of the loaded car, $a\ w\ (\sin \beta'' - \sin \beta') = (W' + W)\ h$ and $\beta'' = \sin^{-1} \dfrac{(W' + W)\ h + a\ w \sin \beta'}{a\ w}$ .....(I)

This would be the true value for β'' if frictional losses were neglected.

Let f represent that part of the total height through which the car falls lost in friction of car travel, and v the greatest velocity of rebound of the car, then $\frac{v}{2}$ = the greatest velocity of the triangle sheave, and $\dfrac{w''\ v^2}{8\ g}$ is the energy lost in the sheave and dissipated in vibration through the timber and triangle.

Then

$$W'\ h = f + \frac{w''\ v^2}{8\ g}, \text{ and } \sin \beta'' = \frac{(W' + W)\left(h - \frac{f}{2}\right) + a\ w \sin \beta'}{a\ w}$$

or
$$\beta'' = \frac{\sin^{-1}(W' + W)\left(h - \frac{f}{2}\right) + a\ w \sin \beta'}{a\ w} \text{.....(II)}$$

The total angle through which the triangle swings is $\beta'' - \beta'$

$$y = r \cos \alpha - r \cos (\alpha + \beta'' - \beta') + \sqrt{m^2 - [c - r \sin (\alpha + \theta)]^2} \text{.....(III)}$$

and when $\theta = \beta'' - \beta'$

$$y = r \cos \alpha - r \cos (\alpha + \beta'' - \beta') + \sqrt{m^2 - [c - r \sin (\alpha + \beta'' - \beta')]^2}$$

Drag of cross-bar then $= 2\ y =$

$$2\left\{ r \cos \alpha - r \cos (\alpha + \beta'' - \beta') + \sqrt{m^2 - [c - r \sin (\alpha + \beta'' - \beta')]^2} \right\} \text{.....(IV)}$$

To determine the weight of the triangle when the drag of the cross-bar is given, substitute the value of the latter in equation (IV), and solve for β''; then substitute the real value for β'' in equation (II), and solve for w.

Second.—From equation (III)

$$\frac{d\ y}{d\ \theta} = r \sin (\alpha + \theta) + \frac{[c - r \sin (\alpha + \theta)]\ [\cos (\alpha + \theta)]}{\sqrt{m^2 - [c - r \sin (\alpha + \theta)]^2}} \text{.....(V)}$$

Equation (V) represents the rate at which y changes with respect to θ or the relative velocities between the two. When $\theta =$

$$\frac{d\ y}{d\ \theta} = \sin \alpha + \sqrt{\frac{r\ m \cos \alpha}{m^2 - m^2}} = \infty .$$

When $\theta = 90° - \alpha$, $O\,A$ is vertical and A is in its highest position, and $\frac{d\,y}{d\,\theta} = \sin 90° = 1$.

Third.—Energy given up to ascending car $= \frac{W\,v^2}{2\,g}$. Energy lost in the triangle sheave $= \frac{w''\,v^2}{8\,g}$, then

$$\frac{W\,v^2}{2\,g} + \frac{w''\,v^2}{8\,g} = a\,w\,(\sin\beta'' - \sin\beta') - 2\,W\cos\delta\Big\{r\cos\alpha - r\cos(\alpha+\beta''-\beta') + \sqrt{m^2-[c-r\sin(\alpha+\beta''-\beta')]^2}\Big\} - Q\,..\text{(VI)}$$

where Q represents that part of the total friction which has already been overcome, and where v is the only unknown quantity and can readily be computed.

Fourth.—Having determined v, since

$$W'\,h = f + \frac{w''\,v^2}{8\,g}, \quad \frac{f}{2} = \frac{W'\,h}{2} - \frac{w''\,v^2}{16\,g} =$$

frictional loss in ascent and descent of the car.

Equations (I), (II), (III) and (IV) indicate that the angular distance through which the triangle is raised, and the horizontal distance through which the cross-bar is carried, are directly dependent on the total weight of the loaded car, the triangle and the fall of the car.

Equation (V) gives the relation of the horizontal velocity of the triangle sheave or cross-bar to the angular velocity of the triangle, and shows that when the sheave is directly over one end of the triangle, or when the effective pull is zero, the relative velocity is infinite. When the end of the triangle is vertically above the axle, then the relative velocity is unity; or the end of the triangle, at this time, when the effective pull is greatest, moves with the same speed as the sheave. In all other positions during the fall of the triangle the sheave moves faster than the end of the triangle, the comparative velocity being represented by equation (V).

By equation (VI) the real velocity of the rebounding car can be determined for any position, either while it is still in contact with the cross-bar or after it has left it.

Construction of the Boom.

The construction of the boom represents a rather uncommon and decidedly interesting problem. Fig. 6 shows the construction of the curves for the 2-ton steam shovel booms. The latter are made in three

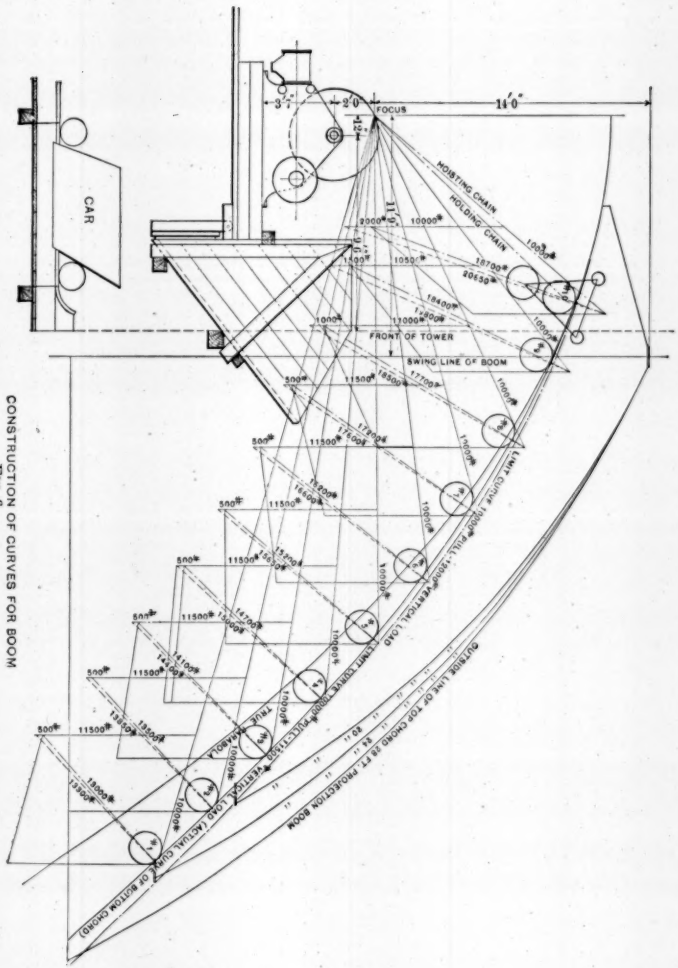


FIG. 6.

CONSTRUCTION OF CURVES FOR BOOM

different standard lengths, so that hoisting may be done at a maximum projection of 20, 24 or 28 ft. The curve of the bottom chords is in all cases the same; the top chords are varied by making them arcs of circles of different radii. The booms of the plant described herein were of 28 ft. projection.

The small circles in Fig. 6 represent the sheaves for the hoisting chain in the boom truck when the latter is at its positions Nos. 1, 2, 3, etc.

The shovel weighs approximately 6 000 lbs., the coal 4 000 lbs., and the boom truck 2 000 lbs., making a total vertical load of 12 000 lbs. The chain pull is 10 000 lbs. If the weight of the boom truck could be neglected, the total downward force would at all times be equal to the pull of the chain, and if the direction of the chain from the truck to the engine were always toward a certain point, which has been assumed as the focus, the sides of the parallelogram of forces would always be equal, and the curve normal to the resultants would be a true parabola having its vertex approximately 12 ft. above the focus. This follows from the property of the parabola that the normal to the curve at any point will always bisect the angle included between the focal chord through that point and a line parallel to the axis of the parabola.

In the construction of these boom curves the chain was assumed to lead from the fixed point on the engine called the focus. This, although not theoretically exact, is sufficiently accurate for practical purposes. To have assumed the focus for each possible tangent position of the chain on the drum would have made the problem much more complicated and would have affected the final resulting curve very little.

If the vertical force exceeds the inward pull by 2 000 lbs., the weight of the truck, and if the curve is constructed so that it is at all points normal to this resultant, it will vary from the parabola, and the result will be the limit curve for 10 000 lbs. pull and 12 000 lbs. vertical load, as shown in Fig. 6. Since these resultants are more nearly vertical than the resultants for the parabolic curve, the 12 000 lbs. limit curve being normal to them, more nearly approaches the horizontal direction. This curve then represents the true theoretical curve, and if the focus is constant, as has already been assumed, the boom truck will be in an exact state of equilibrium during hoisting, neither rising nor falling, but being just on the point of balance.

The booms as actually constructed are made according to a third curve, shown in Fig. 6, about one-third of the distance between the 12 000 lbs. limit curve and the true parabola. In the third curve, the vertical loads are of a varying value. The curve is so constructed, or rather the vertical loads have been so modified, as to take into account the friction of the truck, allowing it to run down readily of its own accord from its highest position (No. 10) when dumping above the hopper. At all other positions of the truck a certain factor of safety exists, whereby a small weight is left to rest on the lower boom chock-block.

This overcomes the possibility of the truck sliding in too freely with the shovel toward the hopper, as a certain remaining downward force keeps the hoisting chain taut during this part of the shovel travel. In position No. 10 of the truck, the weight of the latter has been entirely neglected, and accordingly the curve is there a true parabola. Its grade is greater at this point than it need be, which insures the descent of the shovel with the truck down the boom; whereas with the true theoretical curve the shovel would descend into the hopper, instead of traveling with the truck when the hoisting chain is released.

For positions Nos. 9, 8, 7, 6, etc., the vertical loads have all been modified by amounts determined mostly by experience. The entire curve is somewhat less inclined than the parabola, and more inclined than the true theoretical curve. A curve has been constructed for which the rolling truck friction has been considered, and where a factor of safety is given to the travel of the shovel throughout.

There is another interesting feature about the construction of this curve. Assume, for example, that the shovel is held accidentally in the hatch of the boat, say, by a projecting bolt, or through carelessness of the workmen, or otherwise. In this case the engine may pull its maximum amount, which is 20 000 lbs., the vertical load will be 20 000 plus the truck weight; and since the resultant of this force parallelogram is less vertical than the resultant of the 10 000 — 11 500 lbs. limit curve, or the newly assumed curve, at this point, it has a component in the direction of the track, and the truck will slide a short distance up the boom, which, when noticed by the engineer, warns him to check the engine. There has been allowed in the curve, therefore, a factor of safety, which not only guards the shovel in its travel throughout, but which acts as a safety valve in case of danger.

The curve, although established theoretically, has been verified by the actual workings of the shovel, and the expected results have all been brought out in practice.

Boom Stresses.

It has seemed to the author that in a structure of this nature, where practically every condition exists that is prejudicial to its life, the assumed working strains should be of a most thoroughly conservative nature. The steam shovel is apt to be in operation in very cold weather, and this condition, when members have been heavily stressed, has accounted for the failure of many outdoor structures. Furthermore, there is scarcely a member in the boom which does not receive a repeatedly varying and sometimes rapidly reversed stress.

The short length of the truss emphasizes the need of taking into account this varying and reversal of stress, since the dead-load strains of the structure itself are very small compared with the live-load strains of the traveling shovel. A possible break in the boom is apt to be fatal to the structure as a whole, hence very small fiber strains have been assumed.

When it is considered what careless handling a steam shovel of this kind may receive at the hands of a workman, scarcely any assumed stress can be called too conservative, and where conditions of this kind are to be guarded against, the structure can scarcely be made too secure.

Again, the shovel may possibly be caught in the hatch of a boat and not be perceived by the engineer, in which case the engine would exert its maximum pull, a quantity much greater than that necessary to raise the shovel. To allow for this, all stresses have been computed for the maximum engine pull, irrespective of the loading in the shovel. This, of course, is greatly in excess of the actual working loads. The conditions under which the engine may be exerting its maximum pull are, however, quite possible, and, with a careless engineer and a small hatch, quite probable.

The assumed working stresses, from which the sections of the material have been determined, are as follows: These are based purely on the Weyrauch and Launhardt formulas, adapted to Wöhler's and Spangenburg's experiments. They have been modified slightly to agree more closely with the ultimate strength of the steel used in these structures.

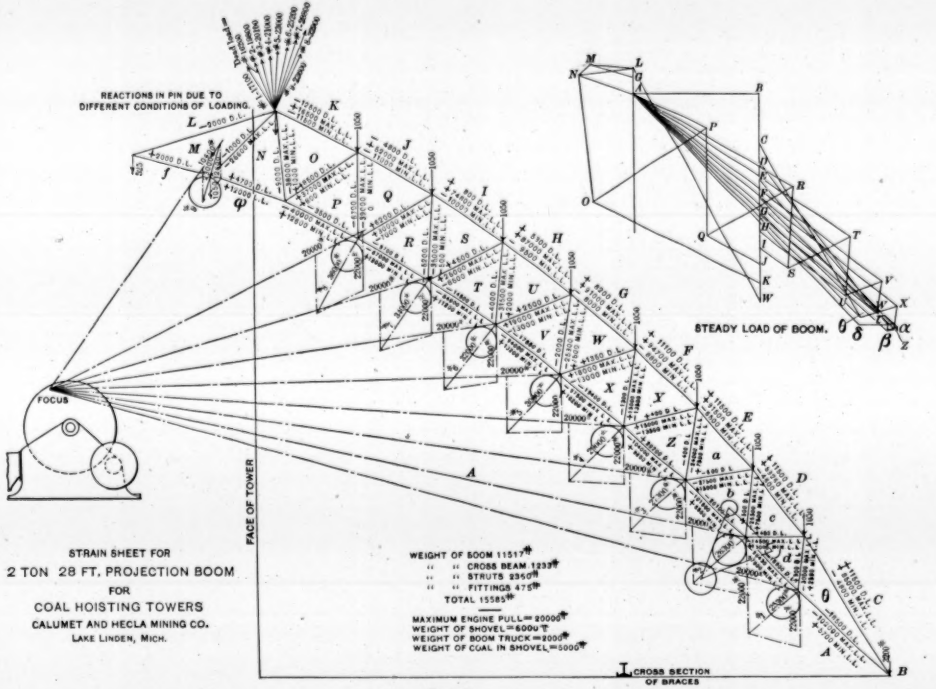

REACTIONS IN PIN DUE TO DIFFERENT CONDITIONS OF LOADING.
STEADY LOAD OF BOOM.
FOCUS
FACE OF TOWER
CROSS SECTION OF BRACES
STRAIN SHEET FOR
2 TON 28 FT. PROJECTION BOOM
FOR
COAL HOISTING TOWERS
CALUMET AND HECLA MINING CO.
LAKE LINDEN, MICH.
WEIGHT OF BOOM 11517#
" " CROSS BEAM 1233#
" " STRUTS 2350#
" " FITTINGS 475#
TOTAL 15585#
MAXIMUM ENGINE PULL=20000#
WEIGHT OF SHOVEL=6000#
WEIGHT OF BOOM TRUCK=2000#
WEIGHT OF COAL IN SHOVEL=5000#

FIG. 7.

Tension.—$8\,000\left(1 + \frac{\text{Min. stress}}{\text{Max. stress}}\right)$ for suddenly applied stresses of the same kind;

$8\,000\left(1 - \frac{\text{Min. stress}}{2\,\text{Max. stress}}\right)$ for suddenly applied stresses of opposite kinds;

$= 12\,000$ lbs. for dead load;

$= 16\,000$ lbs. for wind load.

Compression.—$8\,000 - 30\,\frac{l}{r}$ for live load.

For reversals of stress 8 000 lbs. is decreased as follows:

$$8\,000\left(1 - \frac{\text{Min. stress}}{2\,\text{Max. stress}}\right).$$

For varying loads of the same kind 8 000 lbs. is increased as follows:

$$8\,000\left(1 + \frac{\text{Min. stress}}{\text{Max. stress}}\right).$$

For dead load $= 12\,000 - 45\,\frac{l}{r}$.

For wind load $= 16\,000 - 60\,\frac{l}{r}$.

Rivets.—Loads suddenly applied $\begin{cases} = 6\,000 \text{ lbs. for shear,} \\ = 12\,000 \text{ lbs. for bearing.} \end{cases}$

Wind and dead load.... $\begin{cases} = 7\,500 \text{ lbs. for shear,} \\ = 15\,000 \text{ lbs. for bearing.} \end{cases}$

Fig. 7 is a strain sheet of the boom.

Table No. 1 shows the stresses in all members of the boom for dead load, and for all positions of hoisting, from No. 1 to No. 9.

The reaction at the upper end is taken by the vertical pin shown in Fig. 2. It is always inclined, thus setting up in the pin a bending moment as well as a direct pull. The varying nature of the stress in this pin is shown by arrows pointing in the direction in which the pin counteracts the forces which act on the boom and tend to pull it down; the length of the arrows corresponding to the quantity of stress. The effects of varying stresses on the pin are in this case very much like those usually coming on a steel spring, as they change from one direction to another, inducing alternating tensions and compressions in the extreme fibers of the pin.

The lower reaction is purely horizontal, and is also transmitted into the braces by vertical pins, the stresses varying also as above, but only reversed for one position of the truck, namely, No. 9. Table No. 1 shows the great variation in the stresses of all members.

TABLE No. 1.—Maximum Stresses in the Boom, for the Different Positions of Hoisting.

Member.	Dead load.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.	No. 9.
$C\Theta$	+11 500	+ 85 000	+80 000	+ 72 000	+ 65 500	+ 56 500	+48 000	+34 500	+24 000	− 4 800
DC	+11 500	+ 54 000	+89 260	+ 80 000	+ 73 500	+ 64 000	+53 500	+43 000	+26 800	− 4 600
Ea	+11 500	+ 39 000	+ 64 500	+ 91 500	+ 83 500	+ 72 000	+ 61 500	+52 800	+31 000	− 6 500
FY	+11 100	+ 28 500	+48 300	+ 70 000	+ 94 700	+ 81 000	+69 300	+58 000	+35 000	− 8 800
GW	+ 8 200	+ 20 000	+36 000	+ 52 000	+ 72 000	+ 91 000	+71 700	+62 800	+38 500	− 8 000
HU	+ 5 300	+ 12 500	+25 500	+ 38 500	+ 52 500	+ 69 500	+87 000	+70 000	+44 000	− 9 500
IS	+ 800	+ 3 500	+14 500	+ 23 500	+ 35 000	+ 46 500	+60 500	+74 800	+47 500	−10 000
JQ	− 4 800	− 5 500	+ 2 500	+ 9 000	+ 16 700	+ 25 000	+39 000	+44 000	+52 000	−11 000
KO	−12 500	− 12 500	− 9 500	− 6 500	− 2 000	+ 2 500	+13 000	+13 000	+16 500	−11 500
LM	− 2 000	0	0	0	0	0	0	0	0	0
MN	− 1 000	0	0	0	0	0	0	0	0	−25 600
NO	− 9 000	− 11 700	−13 000	− 16 000	− 20 000	− 23 500	−28 000	−33 000	−38 500	+ 1 000
OP	+ 8 500	+ 10 000	+13 000	+ 16 500	+ 20 000	+ 23 000	+27 000	+32 000	+37 000	+ 800
PQ	+ 6 700	+ 11 000	−13 500	− 16 500	− 20 000	− 23 300	−29 000	−33 000	−39 000	− 0
QR	+ 6 200	+ 10 000	+12 000	+ 14 500	+ 18 000	+ 21 300	+21 000	+30 000	− 7 000	+12 000
RS	− 5 200	− 10 000	−13 000	− 16 000	− 20 000	− 24 000	−25 000	−35 000	− 2 500	− 500
ST	+ 4 500	+ 8 300	+11 000	+ 14 000	+ 17 500	+ 21 500	+25 000	− 8 500	− 5 500	+ 1 200
UV	+ 2 500	+ 7 000	+ 9 000	+ 12 000	+ 16 000	+ 19 000	−13 000	− 9 300	− 7 000	+ 1 200
VW	− 2 000	− 8 000	−11 500	− 16 000	− 20 500	− 25 300	+ 7 500	+ 5 000	+ 4 000	− 600
WX	+ 1 350	+ 7 000	+10 000	+ 14 500	+ 18 000	− 13 000	−11 000	− 7 500	− 5 000	+ 1 400
XY	− 1 300	− 9 700	−13 000	− 20 000	− 26 000	+ 6 500	+ 5 500	+ 2 800	+ 2 500	− 1 000
YZ	+ 400	+ 7 500	+11 000	+ 15 000	− 13 500	− 11 500	− 9 500	− 6 500	− 4 500	+ 1 200
Za	− 400	− 11 000	−17 500	− 24 000	+ 7 500	+ 6 000	+ 5 300	+ 2 500	+ 2 600	− 400
ab	− 600	+ 8 700	+15 000	− 27 500	− 11 600	− 9 500	− 8 500	−11 000	− 4 500	+ 1 000
bc	+ 500	− 15 500	−26 500	+ 7 500	+ 6 500	+ 5 000	+ 5 000	+ 5 500	+ 2 700	− 500
cd	− 1 400	+ 13 000	−14 500	+ 13 000	− 11 700	− 10 800	− 8 500	− 6 500	− 4 500	+ 1 000
$d\Theta$	+ 500	− 30 000	+ 2 500	+ 2 000	+ 2 000	+ 2 000	+ 1 500	+ 1 500	+ 1 000	− 400
ΘA	−18 500	−102 000	−97 500	− 87 000	− 79 300	− 69 000	−58 000	−43 500	−29 000	+ 5 700
dA	−18 900	− 83 000	−98 430	− 88 000	− 80 500	− 70 000	−59 000	−44 500	−29 300	+ 7 000
bA	−20 100	− 64 500	−97 000	−101 500	− 92 500	− 80 700	−68 000	−51 000	−34 000	+ 8 500
ZA	−20 200	− 53 000	−73 000	− 93 800	−104 000	− 90 000	−76 000	−63 000	−38 500	+ 9 200
XA	−19 400	− 43 500	−60 000	− 76 500	− 97 500	−100 000	−84 500	−68 500	−42 500	+10 200
YA	−17 650	− 35 500	−58 000	− 61 000	− 77 500	− 93 000	−94 000	−75 300	−47 000	+10 200
TA	−14 500	− 27 000	−37 000	− 46 500	− 58 000	− 70 000	−84 000	−81 300	−52 000	+11 200
RA	− 9 700	− 18 000	−25 300	− 31 000	− 39 000	− 47 000	−58 500	−67 000	−56 000	+12 000
PA	− 3 500	− 8 000	−13 000	− 15 500	− 20 300	− 24 000	−34 000	−35 000	−40 000	+12 600
$N\Phi$	+ 4 700	0	0	0	0	0	0	0	0	−12 000
Mf	+ 2 000	0	0	0	0	0	0	0	0	0
TU	− 4 000	− 9 500	−13 000	− 16 800	− 21 500	− 26 000	−31 500	+ 2 000	+ 2 000	− 400
Reaction upper pin { Hor.	+ 7 300	+ 12 000	+ 9 300	+ 6 500	.....	.....	− 3 000	− 6 500	− 9 500	− 8 500
Reaction upper pin { Vert.	−14 500	− 16 300	−17 500	− 19 000	−21 000	− 23 000	−25 000	−28 000	−31 500	−22 000
Reaction lower pin	+ 7 355	+ 30 500	+28 400	+ 25 700	+23 300	+ 20 000	+16 900	+12 890	+ 8 380	− 1 850

Taking, for example, such a member as "*A Z*", between positions Nos. 3 and 4 of the boom truck; the maximum live-load stress in this member is a tension of 104 000 lbs., when the truck is stationed at position No. 4. As the truck runs up the boom toward the unloading position, the stress decreases, and finally changes into a compression of 9 200 lbs., when the truck is stationed at position No. 9.

Furthermore, these stresses are not such as are induced in a bridge structure whose loading is more or less of a gradual nature, because in this case the live-load is applied in the truly theoretical way. As soon as the engine hoists, the load is there with its full effect. This condition seems to correspond precisely to Rankine's theoretical live-load. In bridge structures, repetitions occur only after long intervals, during which time the material of the structure has the opportunity to recuperate, as it may be called, or to return to its original molecular state, whereas in a structure of this nature, the truck runs over the boom with considerable speed, averaging one trip per minute.

The web members especially are subject to this fatigue of varying and reversal of stress. Take for example such a member as "*a b*." When the truck is in its position No. 2, this has a compressive stress of 15 000 lbs. live load, making a total compressive stress of 14 400 lbs. As soon as the truck leaves position No. 2, and arrives at position No. 3, the stress in "*a b*" is completely reversed from a large compression to a heavy tension.

In determining the sections of material of the boom it was primarily necessary to observe the variation just mentioned. In addition to this, however, it will be noted that there are a number of stresses of a different nature acting upon any one member of the structure. Take for example a member of the bottom chord "*Z A*"; there is first of all a direct tension of 94 800 lbs., which stress may be increased, or the unit stress decreased, in proportion to its variable nature as determined by the assumed formulas; secondly, the bottom chord, being a curved member, has thrown into it an eccentric stress, producing a bending moment, which tends to increase the tensile strain on the lower fiber of the chord, and decrease that on the upper fiber. A third strain, which is torsional, is induced in the member by the truck which travels on the flanges of the 6 x 6-in. angles of the bottom chord.

TABLE No. 2.—Tests of Structural Steel for Coal Hoisting Towers, Calumet and Hecla Mining Company.

Date.	Specimen.	Size in inches.	Area, in square inches.	Broke at pounds.	Strain, in pounds per square inch.	Limit of elasticity.	Limit of elasticity, in pounds per square inch.	Elongation in 8 ins.	Elongation; percentage of length.	Area of reduced section.	Percentage of reduction.
April 2..	6″ x 4″ x $\frac{7}{8}$″ L	.860 x .636	.547	33 400	61 060	21 800	39 850	1.92	24	.260	52.4
" 2..	Web. 8″ - 18 lbs. I	1.521 x .254	.386	25 900	67 090	17 700	45 850	2.08	26	.180	47.9
" 2..	Bar. 13″ x $\frac{1}{2}$″	1.098 x .507	.557	34 000	61 040	21 700	38 950	2.12	26.5	.254	54.3
" 7..	⊏ Web. 10″ 15 lbs.	1.820 x .254	.462	30 100	65 110	19 800	42 850	2.22	27.75	.211	54.3
" 7..	⊏⊏ Web. 10″ 15 "	1.016 x .256	.260	17 700	68 080	13 400	51 440	1.62	20.2	.131	50
" 7..	⊏⊏ Web. 10″ 20 "	1.410 x .328	.462	30 600	66 230	20 800	45 020	1.86	23.2	.227	50.8
" 9..	⊏⊏ Web. 8″ 11$\frac{1}{2}$ "	1.496 x .202	.302	18 400	60 920	12 100	40 060	2.2	27.5	.157	48
" 20..	L 4″ x 3″ x$\frac{3}{8}$″	1.265 x .378	.478	31 200	65 270	21 030	44 100	2.08	26	.244	48.9
May 2..	I Web. 8″-18 lbs.	1.514 x .248	.375	25 300	67 460	17 400	46 400	1.98	24.75	.192	48.8
" 2..	L 6″ x 4″ x $\frac{3}{4}$″	.740 x .648	.480	31 100	64 700	19 680	41 000	2.06	25.7	.232	51.6
" 2..	L 6″ x 4″ x $\frac{3}{4}$″	.780 x .716	.558	35 400	63 440	23 800	40 480	1.9	23.7	.312	44.1
" 2..	Pl. 13″ x $\frac{3}{8}$″	1.024 x .408	.418	26 900	64 350	16 200	38 750	1.98	24.75	.204	51.2

Note.—Fracture silky, in all cases.

All top-chord members are subject to direct compression and bending moment due to eccentricity of the line of stress. All web members are subject first to a direct pull or compression, and secondly, to a bending moment produced by the tendency of the truck to spread the bottom chords.

It is hardly advisable to go into any of the steps of calculation here; the ways of proceeding are those given generally in books on the mechanics of materials. The reason the writer makes note of these things is that the union of all these stresses, combined with great variation and reversal, is rather uncommon in the ordinary run of structures.

The boom may perhaps be criticised as an uneconomical structure; but when all conditions are taken into account, and the successful working of the boom is considered, it is more difficult to suggest improvements than at first appears.

It is needless to discuss the strain sheets of the towers; these would only represent the results of computations in statics, and would resemble those of a cantilever bridge set vertically on one end instead of horizontally. Stresses throughout the tower are mostly of the same nature as those in the boom, *i. e.*, they are variable and sometimes completely reversed; only in this case the fixed weight of the tower with its engine, hopper, floor, loads, etc., forms a much greater portion of the total straining of the tower, and the immediate shocks from the steam shovel are not so readily felt. Unit stresses, therefore, determined by the assumed formulas, are much larger, and sections of material proportionately lighter.

Under certain conditions it is necessary to apply a downward reaction on the rear posts, and for this purpose heavy straps have been provided for anchoring the towers to the foundation rails.

Table No. 2 shows a few of the actual tests on the material.

The C. W. Hunt Company was the designer and builder of all the coal-handling machinery. The structural steel work of the hoists and coal shed was done by the Passaic Rolling Mill and the Union Bridge Company.

DISCUSSION.

GEORGE HILL, Assoc. M. Am. Soc. C. E.—The paper does not go into the economies of this plant, in that it does not state the capital cost, and therefore the interest and depreciation, as forming parts of the operating expense, are not known. The amount of wages per ton of coal handled, and the amount of fuel burned per ton of coal handled, are not stated. Mr. Hill.

There is no description of the method of generating and transmitting the steam, but judging from the plates, especially Plate III, Figs. 1 and 2, it would seem as though it were generated at some central plant, and transmitted through a pipe line to the traveling tower and to the cylinders of the hoisting engines.

The speaker may be in error, but, to him, this seems to be an exceedingly antiquated method of utilizing the energy of the combustion of coal, and a few calculations he has made seem to indicate a far more economical method of doing the work.

In the first place, to measure the work done, approximately 10 000 lbs. are lifted to a height of about 40 ft., which is 400 000 ft.-lbs.; then 12 000 lbs. are lifted to a height of about 25 ft., which is 300 000 ft.-lbs.; making a total of 700 000 ft.-lbs. The author states that a trip is made in about 50 seconds. Without serious error, this may be taken as one minute. The efficiency of the hoisting engine, apart from the thermo-dynamic efficiency of the steam, would not exceed 40%, which would give about 1 750 000 ft.-lbs. of work done by the engine; or, if it be assumed that the work is done in one minute, about 53 H.-P. The speaker's experience with engines of this type is that their water consumption per I. H. P. is about 150 lbs. Considering the length of pipe through which the steam has to be transmitted, the liability which exists therein for leaky joints and condensation losses, the fact that the engine is being constantly reversed, that it is in an exposed position, and that probably the cylinder drips are always slightly open to take care of condensation, it would seem that 150 lbs. is a very reasonable estimate. This will give about 14 lbs. of coal per minute, or for 2 tons of coal delivered on the pile. If coal is worth $4 a ton, the fuel cost per ton of coal delivered on the pile is 1.4 cents.

If, on the other hand, an electric motor were installed for handling the hoisting drums, the electricity being generated in a compound condensing plant, under cover, in some central location, and the apparatus controlled by some one of the usual and well-tested methods, the fuel cost per ton of coal would be about as follows: In the first place, the mechanical efficiency of the hoisting engine will be about 50 per cent. The work done is 1 400 000 ft.-lbs., and the resulting horse-power

Mr. Hill. is 42. As there are practically no line losses to consider, no losses from cylinder condensation, drips, leaky joints or anything of that sort, it may be said that a horse-power is delivered to the engine or the electric motor with the expenditure of about 30 lbs. per I. H. P.-hour, which gives 2.3 lbs. of coal per minute and a resulting cost for fuel, on the same basis as above, of about $\frac{1}{4}$ cent. So that there is quite a margin within which apparently the economy of the plant could be improved.

So far as concerns the cost of installation, the speaker believes that electric motors and a generator would have been fully as cheap as the steam plant, and the operating expenses would certainly be very materially decreased in the item of wages, for keeping up the pipe line, central station work, etc., as well as in the item of fuel. If the load due to the shovel and trolley was partially counterbalanced, it may be pointed out that whatever was practicable in that respect for the steam hoist would also be practicable for an electric hoist. Consequently the ratios of coal consumption and cost remain practically the same. The speaker does not advance those figures as being absolutely correct, but as being relatively so.

The speaker has no knowledge of any steam line of any length which has no steam losses due to leaking joints, and in such lines condensation losses are at least equal to 5% of the weight of the steam transmitted. Neither has he any knowledge of flexible steam pipe in which losses were not excessive so long as the pipe was flexible. If the transmitting steam pipe was proportioned in size to supply several hoisting towers simultaneously, the losses due to radiation would be proportionate to the size of the pipe rather than to the pounds of steam transmitted, and would, in consequence, be greater. The maintenance of such a pipe line in a proper condition would be an item of expense decreasing in amount as the installation charge increased.

In an electric transmission, on the other hand, there would be no appreciable transmission loss and practically no maintenance charge to be made. If the generating plant be considered, in each case, as independent, for the purpose of operating the unloading towers alone, the total wages account for the electric plant would be less than for the steam plant. If it be in each case considered as a part of a large central station generating power for a number of other uses, the cost of such power at the central station end of the transmitting device would be far less for the electric than for the steam plant.

The speaker uses $4 per ton as a convenient price, but the ratio would hold good for any price. The absolute amount of the saving that might be effected can be ascertained if the author adds to his paper all the items of expense heretofore mentioned by the speaker as being omitted.

O. J. MARSTRAND, M. Am. Soc. C. E.—The coal-handling plant in the free port of Copenhagen has five elevators for 2-ton steam shovels and thirty-six automatic tracks. The coal is stored in three pockets, each having a capacity of 10 000 tons, and in a yard having a capacity of 30 000 tons, situated immediately back of the pockets. Mr. Marstrand.

The plant is owned and operated by the Danish Coal Company. The machinery was designed by The C. W. Hunt Company, and built partly by them and partly by The Titan Company, of Copenhagen, who were the contractors for the entire plant.

With one or two exceptions, the machinery is identical with that described by the author, the differences being introduced to meet local requirements. One of these was that the coal should, at times, be unloaded from the steamers into barges brought up close to and on the farther side of them, which meant that the shovel should be run out to the end of the boom and there dumped. For this purpose a secondary or trolley engine was placed in the tower, its operating lever being brought to the operating stand of the main hoisting engine. A rope, attached to the boom truck, passed over a sheave at the outer end of the boom, to a sheave at the top of the elevator, thence around the drum of the trolley engine and back to the boom truck. In this manner the boom truck could be moved in and out from its position over the hatch of the steamer to a position from which the coal could be dumped through a hopper into the barge.

Another local requirement was that the dock on which the elevators moved should be kept clear for the passage of railroad cars, and that the coal, at times, should be dumped directly into these cars through hoppers in the elevators. The rear legs of the elevators were, therefore, not carried down to the ground, but were supported on wheels on a rail attached to the front walls of the storage buildings, at such an elevation as to leave a clear headway for the cars. The two rails of the track on which the towers moved were thus built at a considerable difference of elevation, and were supported on separate foundations. It should be noted, however, that the coal pockets, to which the upper rail was attached, are unusually substantial structures for buildings of this character. They are built of Monier and Melan construction, and the front walls, particularly, are built of vertical I-beams imbedded in concrete, and stand on pile foundations.

This machinery was erected in 1897, and has been in operation for more than a year.

T. A. COFFIN, M. Am. Soc. C. E.—Attention should be called to some of the sections adopted in the construction of the towers and booms. Mr. Coffin.

The struts which hold the booms out from the tower are of a section which might appear to be peculiar, namely, an I-beam on a channel, both being securely riveted together. By taking moments

Mr. Coffin. about the top point of the boom there is obtained a much larger thrust on these struts than the 20 000-lb. engine pull, and, as they are to receive this live loading directly, they require some attention in their design. When the boom is swung around, these struts, rotating on their pins, pass across the surface of the cross-beam which connects them to the boom. Consequently, only single shear is obtainable at their point of connection, and the strain passes through the lower line of the section, the channel being designed to take the entire strain, and the I-beam holding it horizontal.

This section vibrates very little during hoisting, much less than sections formerly used, and which were composed of two latticed channels.

In order to avoid vibration, which is the disagreeable element in most hoisting towers, tending to make engineers and workmen doubtful of their safety, sections were adopted which promised to insure stiffness as well as strength, in the structure.

All the main members of the towers were composed of latticed channels or beams. These main members were secured at their connections by gusset plates, riveted on either side to the flanges of the channels or beams, thus making every joint a fixed end. Sections plated in that manner take the shake out of the structure, as they are continuous beams.

By comparative estimates of similar towers with others composed of four angles back to back as main members, the weights were found to be nearly the same, but the angle-iron towers had comparatively little stiffness.

In some cases the towers have to be tied down in the rear. This was necessary in this instance, because the tower base was restricted, having only a narrow wharf on which to be operated.

In regard to raising the entire load of 10 000 lbs. for each trip of the shovel, the speaker would call attention to the fact, that, as the weight of the shovel was nearly balanced by a counterweight, the engine had only to raise the coal plus a small fraction of the weight of the shovel.

CORRESPONDENCE.

J. KAHN, Jun. Am. Soc. C. E.—Referring to Mr. Hill's remarks, the steam is conducted to the towers from a central boiler plant situated near the towers. The steam main shown in Fig. 2 leads along the rear of the towers. Steam connections from the engines to the main are made with 3-in. wrought-iron pipe, jointed at intervals with flexible steam joints, which will allow free movement of the towers along the wharf. All pipe is well covered with a non-conducting material, reducing thereby steam condensation to a minimum. Mr. Kahn.

Mr. Hill makes an error in his computations for actual work done by the engine in hoisting the coal. A portion of the weight raised is at all times balanced by the counterweight, and since the weight of the latter is equal to $\frac{8}{10}$ of that of the empty shovel, the actual weight raised before the boom truck is reached is $10\,000 - \frac{8}{10}$ of $6\,000 = 5\,200$ lbs.; and after the boom truck is reached, 7 200 lbs. This, of course, greatly reduces Mr. Hill's approximations of the cost of hoisting.

The question of electric driving was seriously considered before the final arrangement of these hoists was decided upon. The requirements for motors to operate these engines were laid before a number of electrical manufacturing companies, but owing to the irregular working of the engines, together with the large power required, none of these companies cared to undertake the task of equipping the engines.

Briefly, the conditions were as follows: The hoisting engine, working at its normal rate, has a capacity of about 150 H.-P., and its ultimate capacity approaches close to 200 H.-P. It is compelled to change its direction of working twelve times every 50 seconds, the time of one trip. Thus, starting at the beginning of hoisting, it runs forward; slows up as it approaches the boom; stops above the hopper; holding chain drum is here fixed, and hoisting chain drum reverses, allowing the shovel to open; hoisting chain drum is run forward, closing the shovel; engine is stopped for an instant; both drums are reversed, allowing the shovel to descend the boom, at first slowly, then rapidly; holding drum is again fixed, hoisting chain drum reversed, allowing the shovel again to open; holding drum is freed, allowing the shovel to fall into the coal pile; hoisting drum is run forward until the shovel closes; both drums run forward, lifting the filled shovel again toward the boom. All these changes in motion take place every 50 seconds.

It can readily be seen that where so many operations must be performed during such short intervals, it would be a very difficult problem to secure a motor which would work satisfactorily.

Vol. XLI. JUNE, 1899.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 847.

THE APPROACHES AND TRANSPORTATION FACILITIES OF THE PARIS EXPOSITION OF 1900.*

By E. L. CORTHELL, M. Am. Soc. C. E.

PRESENTED FEBRUARY 15TH, 1899.

During a recent visit to Paris, the author became much interested in the engineering features of the Paris Exposition of 1900, and undertook to collect, for presentation to this Society, such information as was obtainable on certain interesting and important points.

The engineers in charge of the several works, fully understanding this purpose, afforded him every facility for inspection, and kindly furnished plans, reports and other data.

The Exposition of 1889 occupied the Trocadero grounds and the Champs de Mars. The special, or rather the grand, feature of that Exposition was the Eiffel Tower, and, as secondary only, the Palais des Machines. These two structures were the only ones preserved. The latter is to be remodeled in its interior by preserving a great central space as a Salle des Fêtes, the remainder of the building being devoted to the Agricultural Exhibit.

* It has been found impossible to reproduce all the plans, drawings, etc., accompanying this paper, but they are all filed in the Library of the Society for reference. A list of the Exhibits will be found at the end of the paper.

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLI, No. 847.
CORTHELL ON PARIS EXPOSITION.

The Eiffel Tower has been kept in use all the time. Recently, various experiments have been made to determine the best color for repainting it. After many trials with various tints it has been decided to paint it sky-blue, in order to make it as inconspicuous as possible, it being thought that, in the clear days which may be expected generally during the Exposition, this color will blend with the blue of the sky and make the tower less prominent.

As it is the desire of the architects to make electricity the main exhibit, and to attract for it the greatest attention, the Electricity Building will be the most striking object to meet the eye of the visitor. The bird's-eye view, Plate V, shows this building standing in the center of the Champs de Mars, in its longer dimension, and near the Salle des Fêtes.

An outline map of Paris is shown on Plate VI.

A greater area for the Exposition than that of 1889 has been provided, by taking the Gardens of the Hotel des Invalides, and also an area on the right bank of the Seine, now occupied by the old Palace of Industry, which is soon to be removed. Two beautiful permanent stone Art Palaces, the larger and the smaller, are nearing completion on this ground. Between them is to be opened a broad avenue leading from the Avenue des Champs Élysées over the river and up to the elevated plateau immediately in front of the Hotel des Invalides. This new avenue will present a striking artistic feature of the Exposition, and will remain as one of the most beautiful avenues of the city. It seemed proper that the bridge to be built across the Seine, to connect the two parts of this avenue, should be made a monumental structure and one of the principal features of the Exposition.

Another quite extensive area has been provided along the left bank of the river, connecting the Champs de Mars and the Gardens of the Invalides—the two main parts of the Exposition on that side. This strip of land which has been reclaimed from the Seine by a very substantial masonry wall, is about 100 ft. wide, and will be occupied mostly by the government buildings of the various countries of the world. The bird's-eye view, while in some respects imaginative, shows how cosmopolitan will be this strip of land. Underneath this strip, and about through its center, has been built a double-track railroad subway, by which the Western Railroad will traverse the entire front of the grounds and reach its large and permanent low-

level station along the Quai d'Orsay and under the Gardens of the Invalides. After reaching the Pont des Invalides, or just before reaching it, the railroad, by reversed curves, passes under the first building marked on the map "Gare des Invalides," and then spreads out into quite wide station grounds, and reaches the opposite side of the Gardens, where it has a duplicate station, both stations being permanent stone structures.

The features thus briefly referred to can be better understood by examining the sketch of the Exposition grounds (Fig. 1), on which the subway above mentioned and the arrangement of the tracks at the Gare des Invalides are shown by heavy dotted lines.

The following works will now be described in the order named:

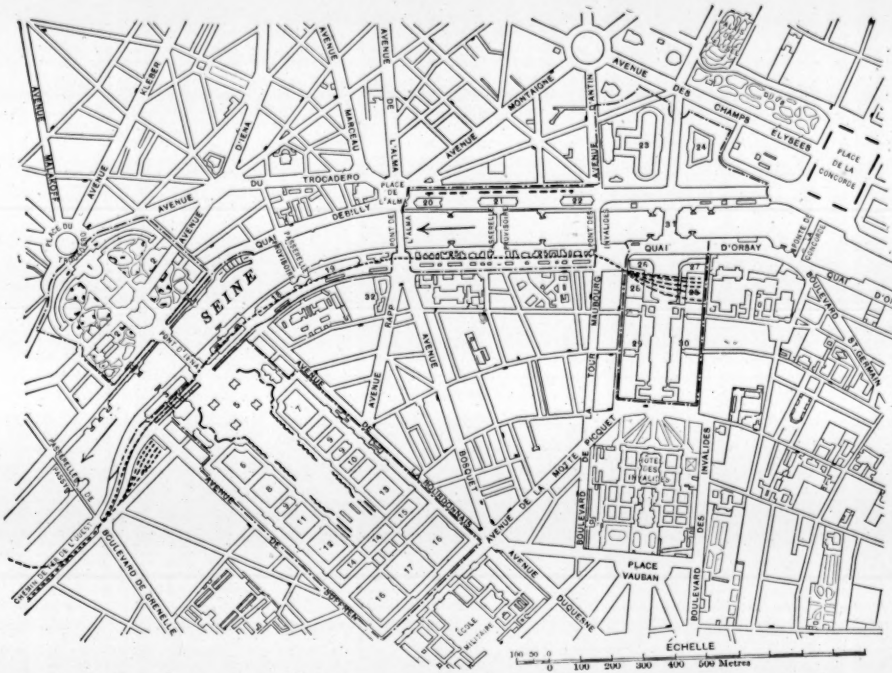
1. The Alexander III Bridge.
2. The Extension of the Orleans Railroad toward the Exposition Grounds.
3. The Extension of the Western Railroad to the Exposition.
4. The Plateforme Mobile (Movable Sidewalk) and Electric Railroad.
5. The Metropolitan Underground Electric Railway.
6. The Enlargement of the Gare de Lyons of the Paris, Lyons and Mediterranean Railway.

The Alexander III Bridge.

As the *Annales des Ponts et Chaussées* contain detailed descriptions of this bridge, it would be superfluous to more than briefly mention this very interesting work. Some of its features are quite extraordinary, and will be worthy of careful study by those specially interested in ornamental bridge construction on an extensive scale.

Mr. Alby, the Resident Engineer, requested the author to present to the Society the photographs shown in Plate VII. These photographs show the elevation of the bridge and a section through one of the abutments, and, though made prior to the construction of the bridge, they are correct, except as to some unimportant features of the abutments, which developments during the preliminary work made it necessary to change.

It may not be generally known that in 1823 a project for a suspension bridge at this location was brought forward and finally approved,



1. Palais du Trocadero.
2. Colonies et Protectorats.
3. Fôrets, Chasse, Pêche.
4. Navigation de Commerce.
5. Tour de 300 Mètres.
6. Aliments.
7. Lettres, Sciences et Arts.
8. Génie Civil.
9. Fils et Tissus.
10. Vêtements.
11. Moyens de Transport.
12. Matériel de la Mécanique.
13. Mines, Métallurgie.
14. Électricitié.
15. Chimie.
16. Agriculture.
17. Salle des Fêtes.
18. Armée de Mer.
19. Armée de Terre.
20. Congres et Économie Sociale.
21. Horticulture et Arboriculture.
22. Ville de Paris.
23. Oeuvres d'Art.
24. Exposition Historique, Art Français.
25. Pavillons des Puissances Etrangères.
26. Education.
27. Enseignement.
28. Gare des Invalides.
29. Industries Diverses.
30. Décoration des Habitations.
31. Pont Alexandre III.
32. Administration de l'Exposition de 1900.

FIG. 1.

and, after two years' work and the expenditure of a considerable sum of money, was nearly finished. This was in 1826. Upon putting the cables in tension and laying the floor of the bridge, the joints in the upper seats of the counterforts of the right bank opened slightly. Some days afterward a water conduit nearly broke in two. No doubt this damage could have been easily repaired, but it gave an opportunity to the opponents of the structure, who were not pleased with its artistic features, the towers obstructing the view of the façade of the Hotel des Invalides, to renew their efforts to have the bridge condemned. It was decided to abandon the work and remove it entirely.

A suspended structure, constructed near by in 1829, transformed in 1854 into a masonry bridge and called the Pont des Invalides, has, until recent years, been ample for the traffic, but, with the increase of traffic, the congestion over the Pont de la Concorde, next above, made it necessary to provide another structure between the two, and the location in extension of the Rue de Constantine was selected, as the Palais d'Industrie blocked the way for one at the location of the demolished bridge.

But the decision to remove the Palais d'Industrie and to build two Art Palaces in these grounds as features of the 1900 Exposition, led to the adoption of the present location, and it was also decided to build a great architectural structure. As is well expressed in the memoire:

"Le choix de l'emplacement de l'Exposition Universelle de 1900, qui a entrainé la démolition du palais de l'Industrie et l'ouverture d'une large avenue entre les Champs-Élysées et l'Esplanade des Invalides, a fait nécessairement abandonner le projet du pont de Constantine et comprendre dans le programme des travaux de l'Exposition une traversée de la Seine dans l'axe de la nouvelle avenue, traversée d'un caractère monumental en raison du rôle qu'elle est appelée à jouer dans un ensemble décoratif."

As is well known, the Czar of Russia, Alexander III, laid the foundation stone of this bridge, on the occasion of his visit to Paris, and this gave it its name.

The plans were under consideration in 1895, by the Minister of Public Works, the Minister of Commerce and the Commissioner-General of the Exposition. The matter was referred to the Conseil-Général des Ponts et Chaussées in June, 1896. The plans of the

OUTLINE MAP
OF
PARIS
METRES
1 200 400 600 800 1000
EXPLANATION
Extension of the Western and Orleans
Railways
Metropolitan R.R, routes

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
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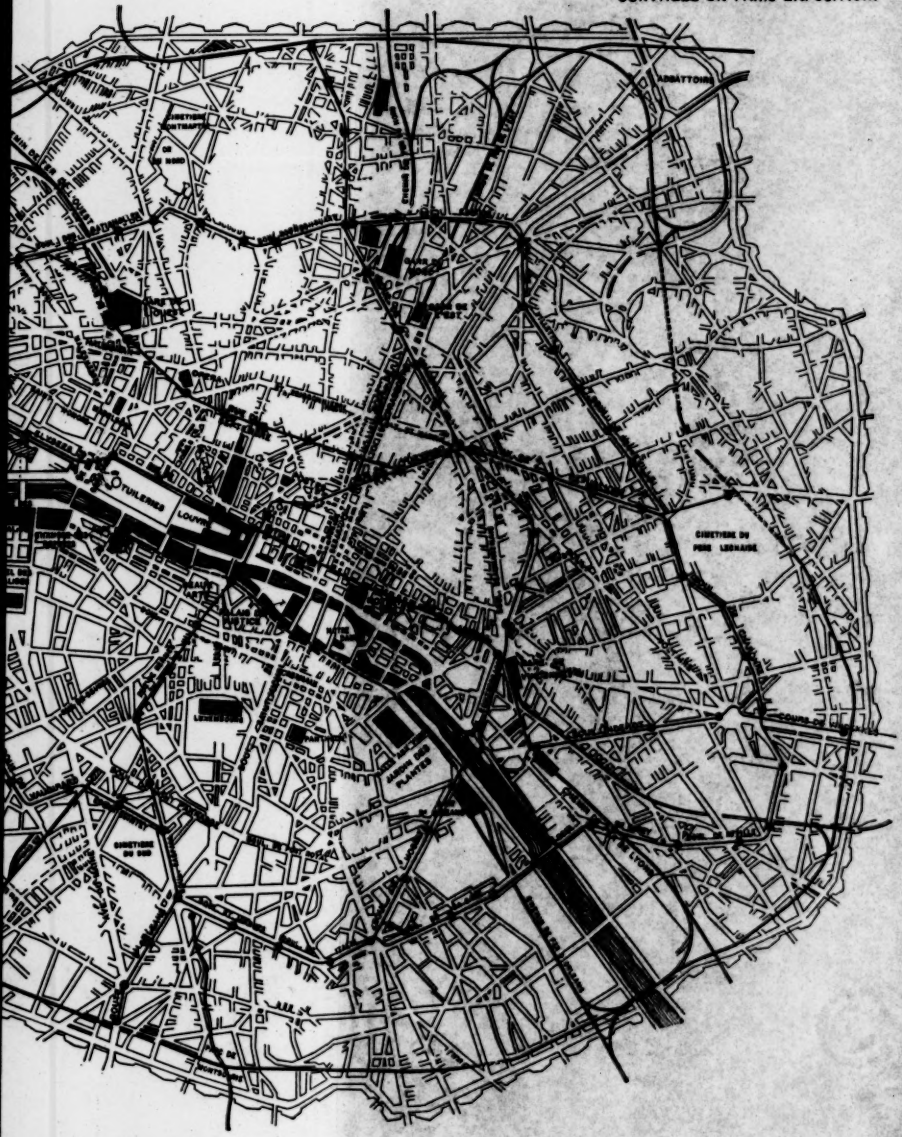
CORTHELL ON PARIS EXPOSITION.



foundations were approved in January, 1897, and of the superstructure in July of the same year.

The architectural features—the decorative parts—were worked out later by the architects, Messrs. Rénard and Cousin, with a view of harmonizing the entire structure with the architecture of the Exposition and other features of the locality. There were three points to consider: *First*, not to interfere with the perspective of the Invalides, viewed from the Champs-Élysées; *second*, not to injure the appearance of the Seine, seen from the Pont de la Concorde; *third*, to make the width in proportion to the width of 100 m. (328 ft.) of the new avenue. On the other hand, the width of clear span for navigation must take into account a bend in the river at the location, which throws the current upon the right bank. To restrict the width at this point would seriously interfere with the navigation by convoy. Very laborious study was given to all these points. In the structure, as finally designed, all these requirements have been met.

The type of the bridge is that of three articulations. The principal features of alignment and dimensions are as follows:

The longitudinal axis corresponds with that of the new avenue, requiring a slight skew with the face of the abutments, which are parallel to the navigable channel of the river.

The width of the river between the two quays is 155 m. (508.5 ft.). The abutments project somewhat, so that the articulations of the two ends are 107.50 m. (352.69 ft.) apart.

The width of the bridge is 40 m. (131.2 ft.) between guards; a central roadway of 20 m. (65.6 ft.) and two sidewalks, each of 10 m. (32.8 ft.). The grade toward the center is 2%, joined by a vertical curve of 32 m. (104.99 ft.) chord and of 800 m. (2 624 ft.) radius.

The curve of the cross-section of the roadway is also of 0.20 m. (0.656 ft.) rise.

The metallic structure is composed of fifteen arches, or ribs, equally spaced. These ribs are of cast steel; the superstructure, floor, etc., are of rolled steel, and the decorative parts are castings.

As the river could not be obstructed an entire year, during the erection of the structure, it became necessary to devise some means of erection to avoid this. For this reason the ribs are united by bolts rather than by rivets, and the erection is performed by means of a traveler which spans the river, moving on wheels, on tracks laid on

each abutment. This traveler, Figs. 2 and 3, moves down stream from each set of two ribs to another set—a veritable rolling bridge.

The flatness of the arch, necessitated by the reasons already stated, is one of the unique features of the structure. The axes of the end articulations are 2.25 m. (7.38 ft.) above the normal level of the water

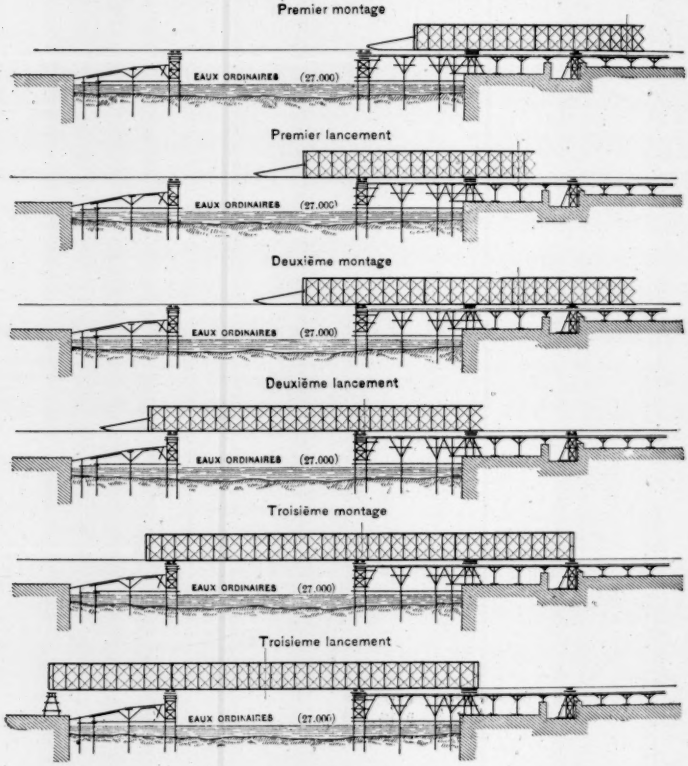
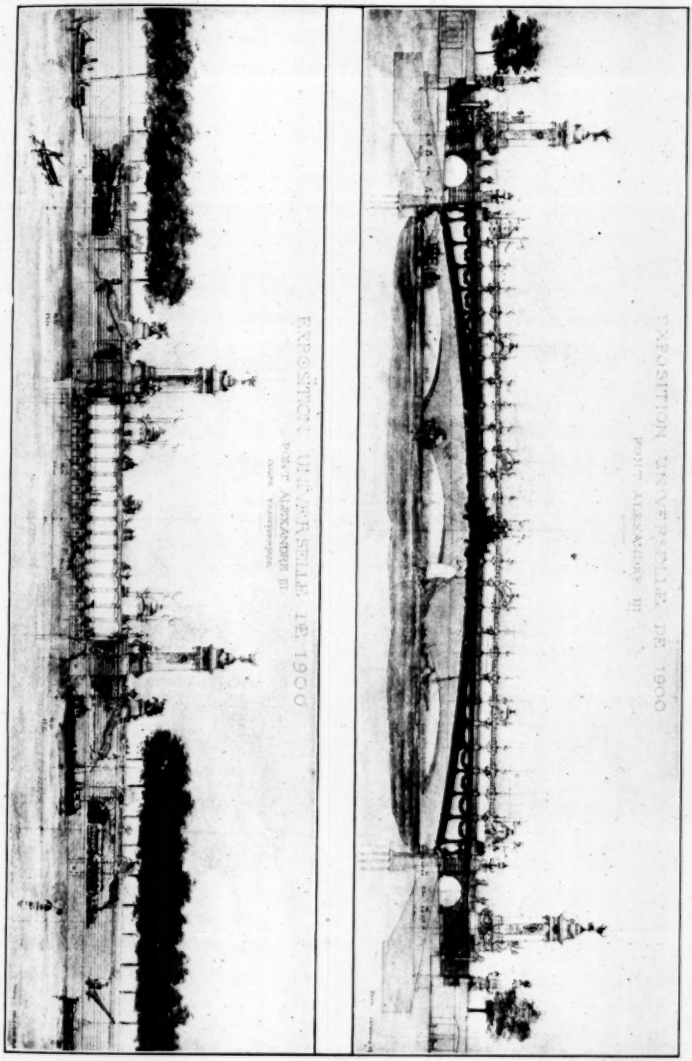


FIG. 2.

and the articulation at the center is 8.53 m. (27.98 ft.) above the same level. The ratio of rise to length is 1 to 17.12.

The foundation work was not only very interesting, but somewhat difficult to manage, on account of the great size of the abutments. Owing to the flatness of the arch, the thrust is very great, 288 tons per


EXPOSITION UNIVERSELLE DE 1900
PONT ALEXANDRE III
EXPOSITION UNIVERSELLE DE 1900
PONT ALEXANDRE III

square meter (10.76 sq. ft.). The thrust by the first plan was 345 tons per square meter, and the ratio of rise to span was 1 to 20.

The endeavor to reduce the pressure has been constant, on account of the anxiety caused by the character of the subsoil, due to the settlements in the other old bridges nearby, built, but afterward removed.

The abutments are parallelograms, 33.50 m. (109.88 ft.) with the axis of the bridge and 44 m. (144.36 ft.) with the axis of the river. Each of these immense abutments was founded on a single caisson, divided into five parts by partitions perpendicular to the direction of the river. Above the working chambers there was a metallic truss-work parallel to the direction of the river. The working chambers, and the truss-work as well, are filled with béton in the proportion of 3 volumes of Seine gravel to 2 volumes of mortar. The mortar is mixed in the proportion of 450 kgr. (992.25 lbs.) of Portland cement to 1 cu. m. (1.31 cu. yds.) of sand. The mean thickness of the béton is 3.40 m. (11.144 ft.). All the upper part of the abutment is of masonry. There are on the face three courses of granite, each 0.50 m. in height, and above them a molded granite course, upon which rests directly the masonry for the skewbacks of the arch. These and other details are shown in Fig. 4.

In the execution of the work it was decided to build and sink the caissons in place without dredging. The metallic structure of the caissons is soft steel. The general features, as well as some details showing the air-locks and shafts, of which there were ten, are shown on Exhibit F 4.* The height of the caisson proper was only 3.68 m. (12.07 ft.). The exterior walls were carried up by prolonging the sides as the caisson sank, as shown in Fig. 5.

The compressed air used in the caissons was taken directly from the pipes of the Popp system near by. The caissons were lighted by electricity from plants established at the works on each bank. The depth of the cutting edge of the caisson below ordinary level of the water was about 30 ft.

The details of the descent of the caissons are graphically shown in Exhibit F 7.* Some details of the granite foundations of the skewbacks, the weights, lines of thrust, etc., appear in Exhibit F 8.*

The ornamental features of the four towers appear in Exhibit F 9.*

* These exhibits are not reproduced herein, but are on file in the Library.

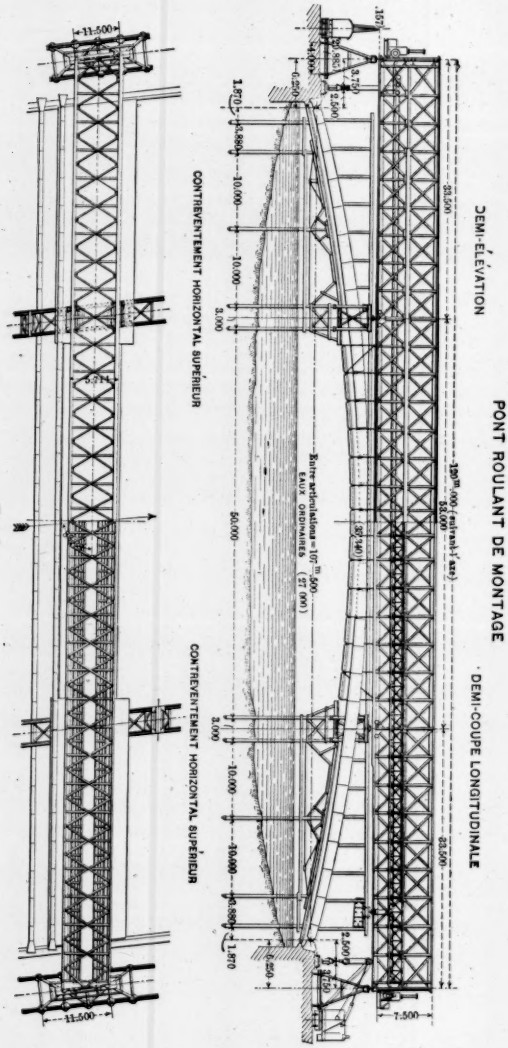

PONT ROULANT DE MONTAGE
DEMI-ÉLÉVATION
DEMI-COUPE LONGITUDINALE
CONTREVENTEMENT HORIZONTAL SUPÉRIEUR
CONTREVENTEMENT HORIZONTAL SUPÉRIEUR
Entre articulations = 107m,500
EAUX ORDINAIRES (27 000)
120m,000 (suivant l'axe)
33.500
33.500
7.500
50.000
11.500
11.500

FIG. 3.

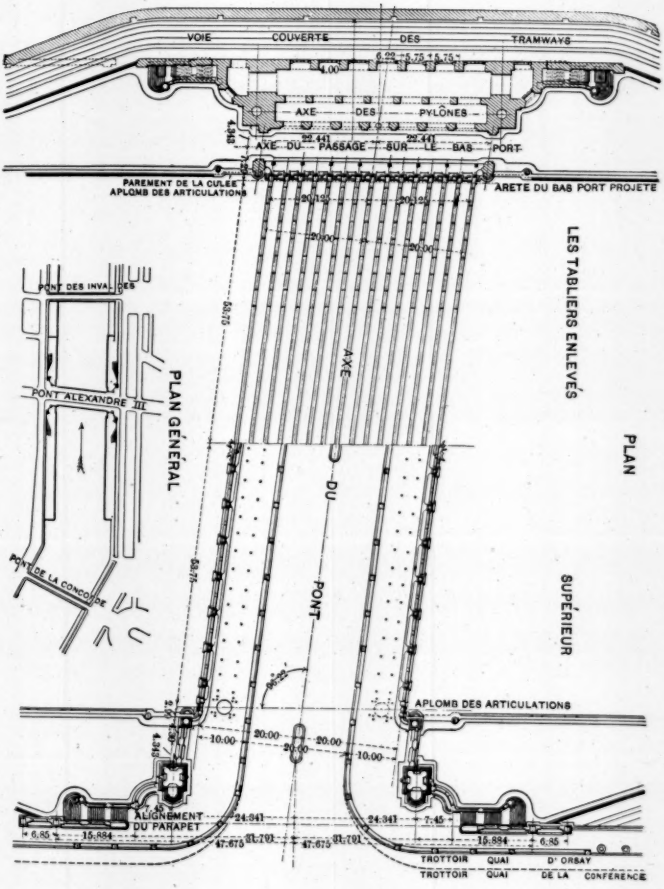

VOIE COUVERTE DES TRAMWAYS
AXE DES PYLÔNES
AXE DU PASSAGE SUR LE BAS PORT
PAREMENT DE LA CULÉE APLOMB DES ARTICULATIONS
ARÊTE DU BAS PORT PROJETÉ
LES TABLIERS ENLEVÉS
PLAN SUPÉRIEUR
PONT DES INVALIDES
PONT ALEXANDRE III
PONT DE LA CONCORDE
PLAN GÉNÉRAL
AXE DU PONT
APLOMB DES ARTICULATIONS
ALIGNEMENT DU PARAPET
TROTTOIR
QUAI
D' ORSAY
DE LA CONFÉRENCE

Fig. 4.

The four sitting figures are to represent the four periods of French history, Charlemagne, Louis XIV, the early and the later Republics.

The engineers, Messrs. Résal and Alby, are from time to time writing the history of the work in the *Annales des Ponts et Chaussées*, and at the recent Fiftieth Anniversary of the French Society of Civil Engineers they gave a general description which will soon appear in its *Bulletins*.

As a matter of interest, in comparing this bridge with the Mirabeau Bridge, further down the river, Mr. Alby has presented photographs of that bridge, with a description of the same which appears as Exhibit I*.

Some of the printed matter from which the above brief description is taken is found in No. 6 and No. 27, pages 165–214 and 311–328, respectively, of the *Annales des Ponts et Chaussées* of 1898. Further descriptions by the same writers are to follow.

The arched platform hanging from the rolling bridge was ready for the erection of the arch members about November 10th, 1898, when the work of placing the permanent arch ribs was begun.

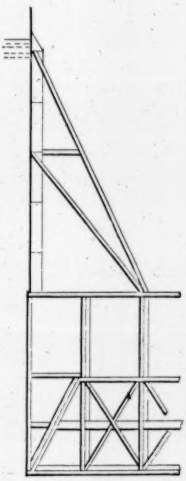


FIG. 5.

THE EXTENSION OF THE ORLEANS RAILROAD TOWARD THE EXPOSITION GROUNDS.

The new station at Quai d'Orsay at the level of the tracks is shown in Plate VIII. Outside of the plans there are, however, a few points to be noted.

Originally, all the Paris railway terminals were in the faubourgs, outside the city proper. To-day the stations of the Northern, Eastern and Western railways are within the city, but it is because the city has grown in these directions. Originally, they were surrounded by cultivated land, and the St. Lazare station, now in the very center of the city, was in quite a picturesque quarter, to which had been given the name "Old Poland."

* These exhibits are not reproduced herein, but are on file in the Library.

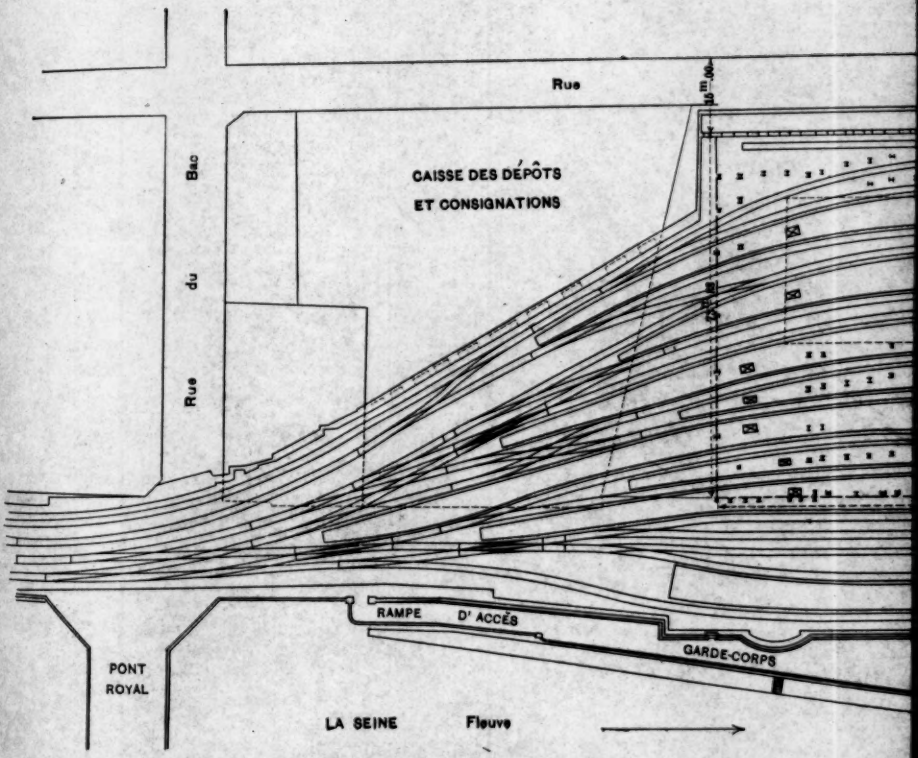
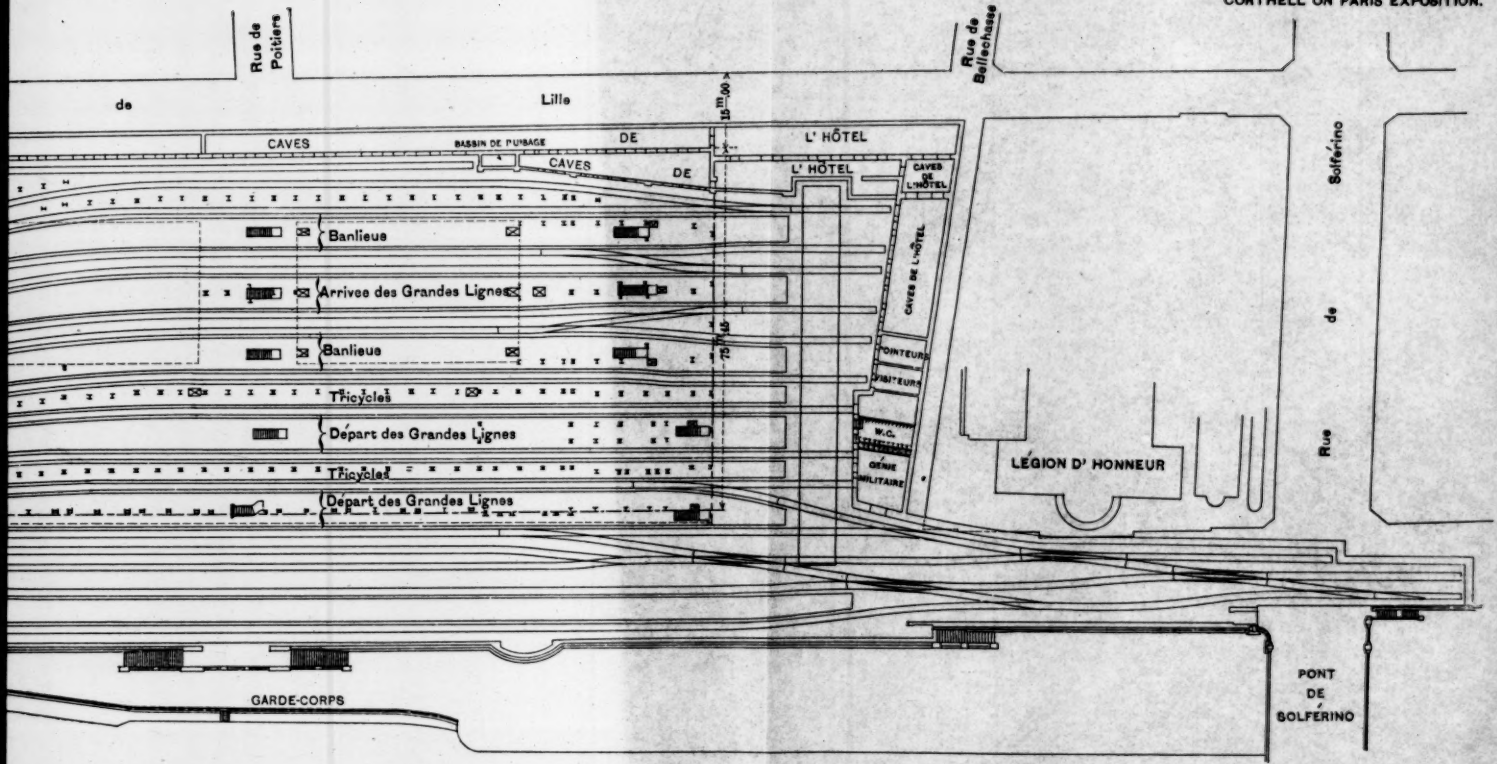

Rue
CAISSE DES DÉPOTS
ET CONSIGNATIONS
Rue du Bac
RAMPE D' ACCÈS
GARDE-CORPS
PONT
ROYAL
LA SEINE
Fleuve

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLI, No. 847.
CORTHELL ON PARIS EXPOSITION.



Mr. Brière, Chief Engineer of the Work, in some remarks before the French Society at its recent Fiftieth Anniversary, stated that great cities generally grow toward the north and west. The terminal station of the Orleans Railroad was in the east and south, in a region where the population has not developed. It was obliged to approach the center, and was encouraged to do so by the success which attended the subway extension of the Chemin de Fer Sceaux to the Luxembourg station. The old terminal station of this line had 3 000 000 passengers in 1894, the new station had 4 800 000 in 1896, and 7 000 000 in 1897.

The difficulty with the Orleans extension was to find for the station a place which should have about 30 000 sq. m. (322 900 sq. ft.). However, a suitable location was found by purchasing and removing the ruins of the old "Cour des Comptes," which had been a caserne or barracks. At the side of it was another State building, "La Casse des Dépôts et Consignations," which could neither be moved nor disturbed. It was therefore necessary to go under it. The author has, with the resident engineers in charge, examined this difficult work, by which an extensive area under the corner of this building is being prepared for the tracks branching out into the subterranean station.

The whole line is about 4 km. (2.48 miles) in length. In the map of Paris, Plate VI, this extension is shown by a heavy dotted line. The map of the Exposition, (Fig. 1), shows on the margin the location of the terminal station, which is about as far from the Gare des Invalides as the Opera House is from the Madeleine.

The extension traverses the present terminal station. The grades to reach the subway are very light. For the entire distance the extension is along and adjacent to the quay walls of the Seine, directly under the quay streets and very near to the surface of these streets, often not more than 18 ins. below them.

Generally, the municipal government has prohibited open cuttings in the streets and covering them afterward with metallic floors on columns. Therefore, tunneling is required, and this is being done by means of a hydraulic shield. It is noticeable, however, that in several places the pavement was disturbed to such an extent that, while work underneath was going on, it was necessary to guard the areas with ropes to keep off vehicular traffic, and that the surface had to be raised and repaired. Two typical sections are shown in Fig. 6.

The rails are laid at about the ordinary level of the water of the

Seine. In floods the water rises nearly to the level of the top of the subway, sometimes 5 m. (16.4 ft.) above the ordinary level. As there is a comparatively thin wall between the subway and the river, it is necessary to make a tight dam of this wall, and also, generally, to lay

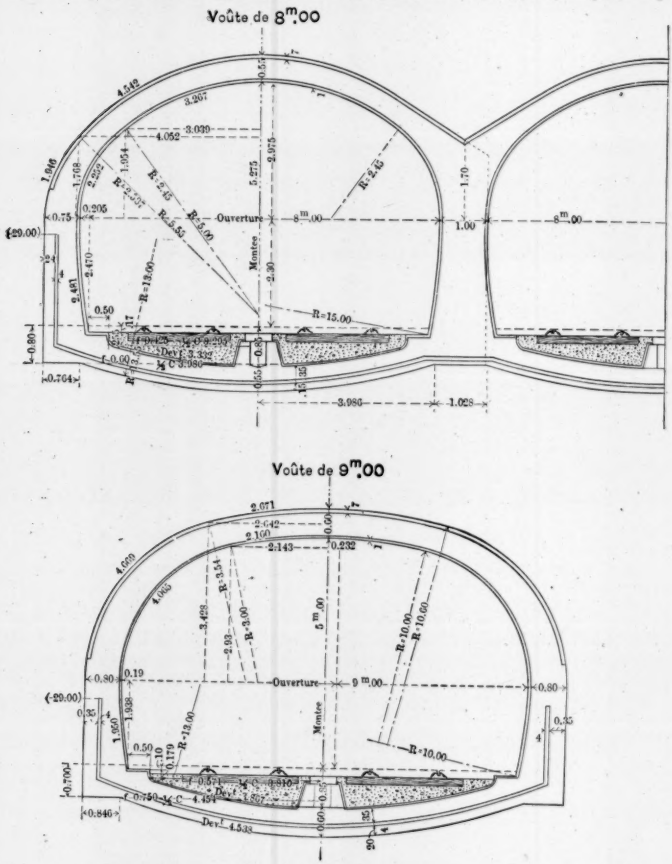


FIG. 6.

a very thick invert of béton to resist the upward pressure and the percolation in times of flood. Probably it will be necessary to install pumping plants for these occasions.

At one point, where new quays for river passengers and merchan-

dise had to be provided, the work was done in open trench for about 650 m. (2 132 ft.). Of this work, approximately 55% has a masonry arch, 31% has a metallic floor, and 14% is in open trench. (These proportions are somewhat in doubt.) In the open-trench work the width of the cross-section is 9 m. (29.5 ft.), 8 m. (26.24 ft.) being for the tracks and 1 m. (3.28 ft.) for the employees. At the junction with the Sceaux Railroad, which had to be provided for, there are four tracks, in two tunnels, each 8 m. wide. Elsewhere there are two tracks. The two tunnels communicate with each other by wide bays. For aeration, and also for light, the walls of the Seine are pierced above the flood level.

There will be one intermediate station between the present terminal and the new station at the Quai d'Orsay. It will connect with the banlieue line (ceinture), and will be about 11 m. (36.09 ft.) wide.

In the new terminal station, Plate VIII, there will be fifteen tracks, common to arrivals and departures. The shortest platform is 240 m. (787.4 ft.) long, and is at the level of the platforms of the cars, 88 cm. (34.64 ins.) above the rails. The baggage will be handled on separate platforms, which are 3.50 m. (11.48 ft.) wide, with elevators for lifting or lowering the baggage. The tracks at the further end of the station are all connected by turn-tables, 6.20 m. (20.33 ft.) in diameter. The length of the ground floor for passengers is 200 m. (656 ft.). All the departure service is done on the Quai d'Orsay, the arrival service on the Place Belle-Chasse. It is impossible to provide a court for departures, but a widening of the space is made. The arriving passengers, on ascending to the ground floor from the level of the platforms, will find a carriage court 100 m. (328 ft.) long and 50 m. (164 ft.) wide, where the carriages are stationed.

Over the station there is to be a hotel of considerable proportions. It will have a dining room of 420 sq. m. (4 519.2 sq. ft.), a salon 300 sq. m. (3 229 sq. ft.), galleries of 150 sq. m. (1 614 sq. ft), a grande salle des fêtes of 450 sq. m. (4 842 sq. ft.), and 275 chambers. The entire structure is carried on metallic supports. It was necessary to divert a great many sewer, water and gas pipes, etc. One intercepting sewer, about 4 m. (13 ft.) in diameter, that passed directly through the new station ground, had to be diverted, at a cost of 5 000 000 francs ($1 000 000). This sewer received all the water of the left bank and conducted it to the siphon of the Place de la Concorde, under the Seine.

The traction will be electric. The engineers and experts visited the United States and made special examinations of the electric tunnel line in Baltimore. They returned with the conviction that it would be possible, easy and economical to make an application of electric traction with a full-sized train, such as run from Paris to Bordeaux, and on the Circular Line of Paris.

They are, however, still studying the question, but everything now favors the use of separate special electric motors, changing from the steam locomotive at the Quai d'Austerlitz, the present terminal for the extension to the Quai d'Orsay.

The cost of the entire work will be about 40 000 000 francs ($8 000 000), or 10 000 000 francs per kilometer. The expenses are divided about as follows:

Personnel	1 000 000	francs.	$200 000
Land	13 000 000	"	2 600 000
Disturbance of streets, etc	1 000 000	"	200 000
Sewers	5 000 000	"	1 000 000
Water and other conduits	500 000	"	100 000
Principal construction	10 500 000	"	2 100 000
Tracks and ballast	1 000 000	"	200 000
Buildings	8 000 000	"	1 600 000
Total	40 000 000	francs.	$8 000 000

The work will no doubt be completed before the opening of the Exposition.

The Extension of the Western Railroad to the Exposition.

The route of this improvement and extension is shown on the map of Paris (Plate VI). The terminal at the Champs de Mars is shown on the map of the Exposition (Fig. 1).

From the St. Lazare Station to Station Courcelles, which is a junction station of the belt line, there are four tracks; from this station only two.

The great additional traffic of the Exposition has made it necessary to place two more tracks from Station Courcelles to the point in Passy, where the new line, also a double-track line, is to leave the main line for the Exposition. Some features of this four-track work are quite

ENCORBELLEMENTS EN BETON ARMÉ

SYSTÈME HENNEBIQUE

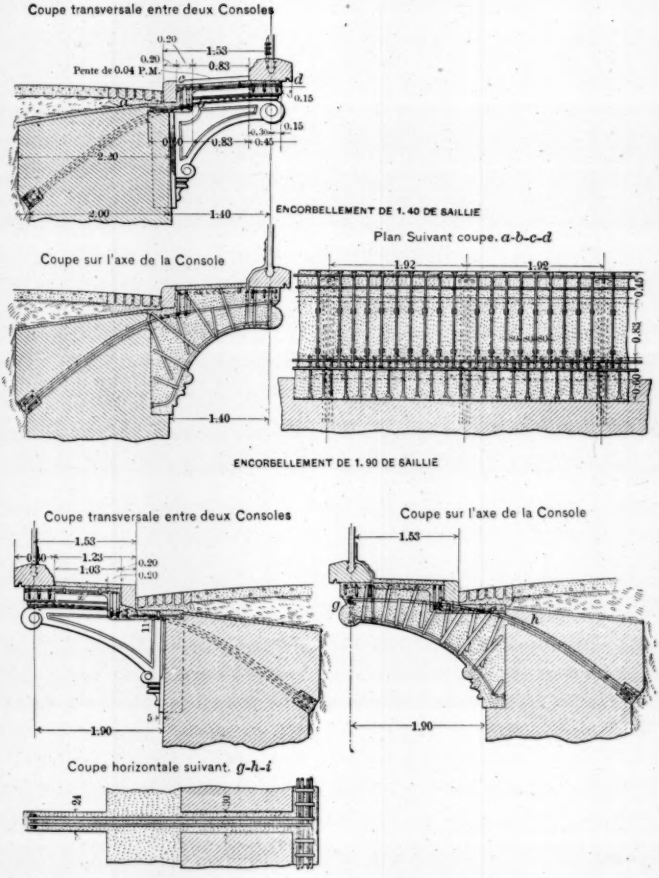


FIG. 7.

difficult, requiring veritable tunnels under such avenues as the Bois de Boulogne and Grande Armée. It should be understood that the entire line from the St. Lazare Station is depressed. Hitherto, from Courcelles to Passy, there was a simple excavation, with the usual earth slopes and with a railing on the edge, as the line runs through an avenue.

To accomplish the widening it was necessary to build nearly vertical retaining walls. As this work infringed on the sidewalks in some localities, particularly at Station Courcelles, the plan of projecting sidewalks has been adopted. The peculiar method used is illustrated in Fig. 7. On buildings, such work had previously been done by architects, but it is believed by the railway engineers that this is the first instance of its use on railway work. It is simply a concrete sidewalk of varying width, resting on concrete brackets of ornamental shape anchored into the retaining wall by round rods extending from the front of the bracket through it to take hold of similar rods, which are imbedded in the wall, and run lengthwise with the sidewalk. Strap-iron is also used to connect the upper and lower rods of the brackets. The maximum overhang to the line of the railing, or fence, is 1.90 m. (6.22. ft.)

The city authorities require very severe tests to establish the absolute safety of the sidewalk. A heavy weight is dropped several feet and several times upon the outer part of the greatest overhang. No starting of the work has yet been noticed under these tests.

At the point of departure of the Champs de Mars Line the main line is in a very deep excavation. In order to avoid a level crossing, the right-hand track of the extension is carried under the tracks of the main line. This requires deep tunneling, and both lines, after they come together from the single tunnels and enter the double-track tunnel, are still carried deep under the surface of the ground to near the Seine.

It will be seen from the plan and profile,* that the line, after coming into the open, curves to the left in crossing the river. The trusses of the bridges and viaducts are quite well shown on the profile; the longest span, that over the navigable way, is quite artistic. All the work is well advanced and there appears to be no good reason why it will not all be ready for traffic before the opening of the Exposition. At the Champs de Mars station it was necessary to buy considerable

* These exhibits are not reproduced herein, but are on file in the Library.

property and demolish quite a number of houses to obtain the room required. After the Exposition, this station, with its numerous tracks, will be used for a general freight yard, and it may possibly become the terminus of a line from this point to Versailles, proposed by the company. The cost of the work being done by the Western Railroad will be about 25 000 000 francs ($5 000 000).

The Plateforme Mobile (Movable Sidewalk) and Electric Railroad.

It was not the original intention to make the entire circuit of the grounds, but by examining the sketch of the grounds, Fig. 1, and the small diagram of the movable sidewalk, Fig. 8, it will be seen that

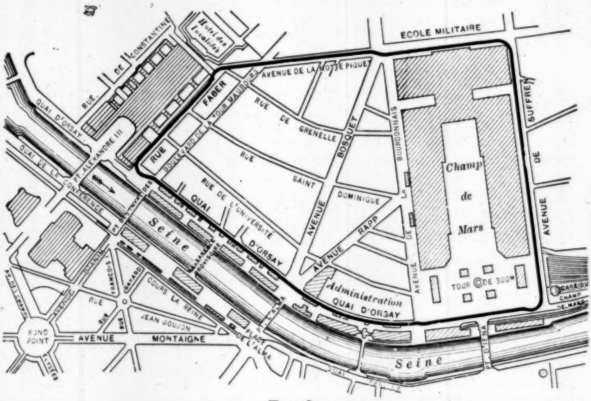


FIG. 8.

the later plans require an entire circuit. The following is a translation of extracts, nearly entire, of three articles in *Le Génie Civil,* and the illustrations are taken from these articles.

These descriptions give the entire matter so fully that it is unnecessary to dwell upon the subject.

The author has obtained from the office of the President of the company organized to exploit the "plateforme mobile," the information that the whole matter is in an experimental condition; a short section of 300 m. (984 ft.) is now being installed for trial at St. Ourc, just outside of Paris. A demonstration of the practicability, safety and convenience of the plan is required by the Exposition authorities, by

the terms of the concession given to the Engineer, M. de Mocomble and his associates. It will appear from an examination of the plans that they differ essentially from those of the Chicago movable sidewalk, which is so well known that a description here is unnecessary. Whether the defects attributed to it by the Paris engineers are real or important will not be discussed, and whether the new plans to remedy them may not possess defects as serious will be demonstrated by the experiments, or at least by the actual transportation at the Exposition.

"*Project for a Moving Electrical Sidewalk for the Transportation of Passengers in the Interior of the Exposition.*—It was not until 1893 that there existed a practical test of the above idea. This was the in-

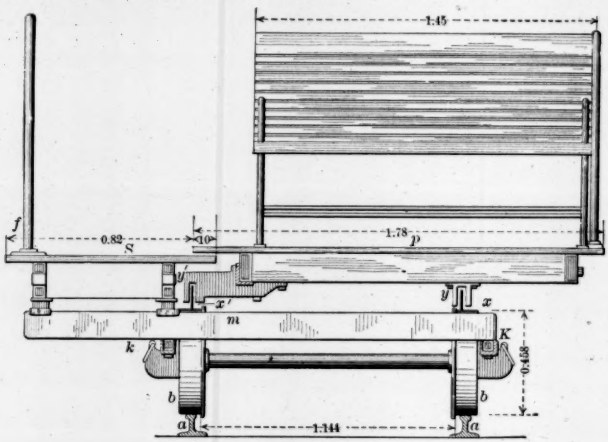


FIG. 9.

stallation by two American Engineers, Messrs. Silsbee and Schmidt, of the first movable sidewalk at the Chicago Exposition. This sidewalk had two rates of speed; that is to say, it consisted of two parts, one moving at a speed of 5 km. (3.1 miles) per hour, and the other at a speed of 10 km. (6.2 miles) per hour. It is certain that in this system it would not be more difficult to go from the first platform to the second than it would to go from the stationary sidewalk to the platform moving at the rate of 5 km. an hour. The method employed to obtain two different rates of speed consisted in placing the second platform upon the rims of the wheels of the first platform by means of flexible rails. Thus, by the same movement, two rates of speed were obtained, of which one was double the other (Fig. 9).

"This system served very well at the Chicago Exposition in 1893, and at that of Berlin in 1896, which may be considered the time of its real practical demonstration.

"*Movable Sidewalk of Messrs. Blot, Guyenet and Mocomble.*—In spite of the good results that it has given, the movable sidewalk of Messrs. Silsbee and Schmidt presents, from a technical point of view, certain defects which have attracted the attention of Messrs. Blot, Guyenet and Mocomble, construction engineers of Paris. One principal defect consists in having all its machinery in motion, which makes the care very difficult and does not permit repairs unless the entire machinery is stopped. Another objection is that the moving rails supporting the second platform constitute, together with the axles and the machinery, a dead weight put in motion uselessly, and, the system not riding easily, the movement lacks smoothness. Finally, as there is only one moving truck in thirty-six, the moving carriage must be ballasted to aid the adhesion, and thus the system does not in reality have its load

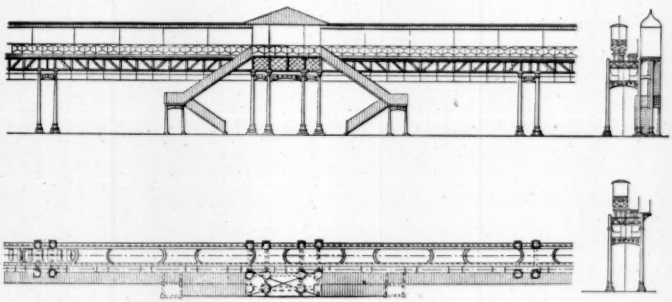


FIG. 10.

uniformly distributed, but is a system of traction by locomotives placed at intervals in a series of cars forming a continuous train.

"In the moving sidewalk proposed for the Exposition of 1900, Messrs. Blot, Guyenet and Mocomble are endeavoring to remedy the faults enumerated above, and we shall endeavor to describe clearly the solution at which they have arrived.

"The whole of the projected installation comprises:

"*First.*—A platform at a higher rate of speed 1.30 m. (4.26 ft.) wide rolling on a track with a gauge of 0.90 m. (3 ft.) at a speed of 2.70 m. (8.85 ft.) a second, that is 9.720 km. (6 miles) an hour (Figs. 10 and 11).

"*Second.*—A platform at a lower rate of speed, 0.80 m. (2.62 ft.) wide, rolling on a track with a gauge of 0.45 m. (1.47 ft.), at a speed of half that of the first (Fig. 10).

"*Third.*—A stationary sidewalk 0.90 m. (3 ft.) wide.

"*Fourth.*—A metallic structure supporting the whole, and resting on metallic columns.

"*Fifth.*—Stairs for the ascent and descent of passengers.

"Each of the platforms is composed of trucks belonging to two distinct types. The first is supported by four independent wheels, while the second has no wheels, and rests on the neighboring trucks. The floor of each truck rests on wooden beams carried by cross-pieces which are fastened at the center to an axial girder, which is the core of the system. These cross-pieces carry at their extremities the wheels which rest on an iron track fastened to the wooden beams carried by the metallic cross-pieces, which are bound to the main girders.

"The axial girder rests, every 6 m. (19.6 ft.), on rollers, sometimes moving and sometimes for support. It consists of plates and angle-bars, with a steel rail in the form of an inverted T (Fig. 11), that is to say, having the flange at the bottom. In order to permit it to pass the curves this girder is divided into sections of 4 m. (13.1 ft.) joined together by hinges.

"The motion is produced by the adhesion of the axial girder on the moving rollers, the same axle-tree being used for both platforms. In order to remedy the sagging which would take place between the rollers, these are mounted on a short axle, held solidly between two bearings as close together as possible and resting on an elastic frame. These two bearings are united by an axle-tree and two Cardan joints, thus allowing for every vertical displacement of the rollers.

"The moving rollers are placed 39 m. (127 ft.) apart, and the supporting rollers 6 m. (19.6 ft.) apart. The motive power, furnished by an electric current, being thus distributed to a large number of points, it is not necessary to overcharge to induce adhesion at the motive points. On the other hand, the constructors have taken all precautions, so that the movement may be as quiet as possible. The wheels of the trucks, as well as the rollers, are covered with rubber, thus deadening the shock and the noise, and we might suggest wheels of compressed paper to do away with all noise.

"Iron posts 1 m. (3.28 ft.) in height have been fastened to the floor of the trucks of the first platform to serve as a support to inexperienced travelers in passing from the stationary platform to the moving one. In going from this latter to the fastest platform, the backs of the seats will be of assistance. Every 200 m. (656 ft.) there are stairways for the ascent and descent of passengers.

"In the arrangements everything has been planned to insure a continuous and uninterrupted movement. To this end the wheels of the trucks, the moving axles and the axial girders have been calculated in such a way that one portion might be missing without making it necessary to stop, or that the neighboring portions should suffer.

"The moving platforms and the stationary sidewalk rest on main

PROJET DE PLATEFORME MOBILE ELECTRIQUE ET A DEUX VITESSES.

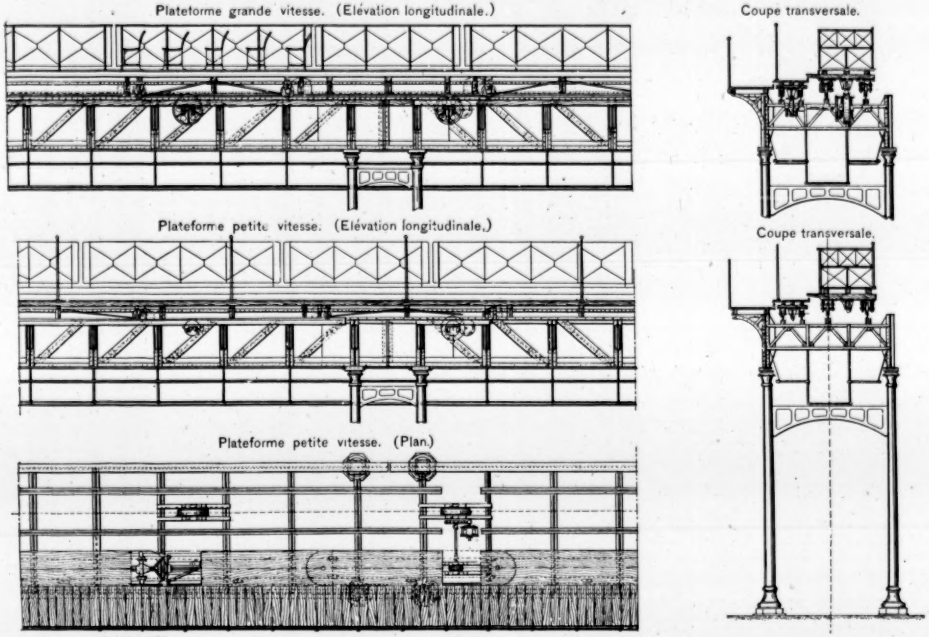


FIG. 11.

girders, braced 1.20 m. (4 ft.) apart, and supported by iron open-work columns, spaced at 19.50 m. (64 ft.) from axis to axis.

"This aerial disposition of the platforms surely presents very alluring inducements, for it permits a complete tour of the Exposition and furnishes travelers with extremely varied panoramic views.

"Unfortunately, these advantages are too dearly bought, for the aerial platform brings with it serious inconvenience. Far from allowing passengers to get on at all points, as would a platform on the ground, it is necessary to make stations, where great crowds would collect, and, by the consequent ascending of stairs, it deprives passengers of a part of the advantages it gives them. And more, and this is without doubt the most serious difficulty, this aerial platform, although light and elegant as it is expected to be, would form a line cutting all perspectives. It is to be feared that it will have against it the opinion of most of the artists called to give their advice, and who would prefer, perhaps, to deprive visitors of a very commodious and agreeable method of transportation rather than admit a construction capable of breaking more or less the harmony of the complete whole which they purpose creating.

"This said, one must recognize that the installation presented by Messrs. Blot, Guyenet and Mocomble is a project carefully studied and perfectly possible of realization, at least from a technical point of view.

"The projected course (Fig. 8) would pass by the Avenue de La Motte-Piquet, l'Esplanade des Invalides, the Quai d'Orsay and Avenue de Suffren, thus forming a closed circuit of about 4.200 km. (2.6 miles).

"The originators of the project, such as has been described, estimate that this platform could take care of a traffic of 50 000 passengers an hour, allowing, on one hand, four passengers per running meter, which would give at first: 4×2.70 m. $\times\ 3\ 600 = 38\ 880$ places, and, on the other hand, most of the passengers, in place of going the whole course, will not go more than two or three kilometers, which will leave about 25% of disposable places, and would increase the number of passengers an hour to $38\ 880 \times \frac{100}{75} = 51\ 840$.

"Even when these figures are submitted to a proportionate reduction, one is struck with the great capacity for traffic that such an installation would possess. This capacity could also be increased in such a manner as might be deemed necessary by a simple increase in the width of the high-speed platform, the other parts remaining the same, or being but slightly modified.

"Messrs. Blot, Guyenet and Mocomble place the cost at about 6 000 000 francs ($1 200 000) for the initial setting up of their installation. They add to this sum about 1 500 000 francs ($300 000) for the

running expense during the Exposition, and for the interest on the capital, and show that, with these conditions, in order that the enterprise be remunerative, it would be necessary to carry only 15 400 000 passengers at the uniform price of 0.50 franc (10 cents). If it is assumed that the duration of the Exposition will be 180 days, that would make a little less than 85 000 passengers per day. The Decauville Railroad of the Exposition of 1889, whose course was very much shorter, and whose traffic capacity was much more limited, carried on an average 33 640 passengers a day.

"The calculations relative to the exploitation have been based upon the supposition that the electric current will be furnished by one of the sectors of the City of Paris at the price of 0.06 franc (1.2 cents) per hecto-watt-hour. Also, a study of the conditions through which the platform will pass has led the constructors to admit that, fully charged, the resistance to the movement will be 5 kgr. (11 lbs.) per running meter, so that the whole of the installation will require $\frac{4\,200 \times 5}{75} = 280$ H.-P. measured by the moving rollers. Allowing only 80% for the estimated power furnished at the receivers and 10% for leakages on the line, the necessary force at the power station would be about 400 H.-P., reduced to 170 H.-P. for the unloaded platform.

"It should also be remarked that the starting would occasion an extra effort of about 20%, but this increase of labor would last only a few seconds a day, and it is not necessary to take it into account in the calculations relative to the expenses of the exploitation."

"This project, undertaken with the technical collaboration of Mr. Henri Maréchal, Engineer of the Ponts et Chaussées, is backed by the General Traction Company and the International Bank of Paris.

"It comprises a single-track electric railroad, having a gauge of 1 m. (3.28 ft.), and an electrical platform at two rates of speed (Plateforme mobile).

"The railroad follows in design the stipulated course, but some modifications in the specifications have been required, namely, that the cross-roads, Rapp-Bosquet, be passed on a viaduct, and that the carriages be 3.15 m. (10.1 ft.) high and 2.50 m. (8.2 ft.) wide.

"The above will comprise eight trains in use and three in reserve, each having 250 places. Each train will require 140 H.-P. The current will be distributed the entire length of the track by a central rail.

"The contractor requests that the power station producing the electricity be placed within the confines of the Exposition.

"The platform, which will make the circuit in an opposite direction to the railroad, is the Blot, Guyenet and Mocomble system, recently described in *Le Génie Civil* (as above). The whole comprises a fixed sidewalk, a first platform moving at the rate of 5 km. (3.1 miles) an hour, and a second moving at double that rate of speed.

"The platform would be continually on a viaduct. Parallel or placed above the railroad, Avenue de la Motte-Picquet, Rue Faber and Quai d'Orsay, it would cross the Avenue de La Bourdonnais, and continuing, on this Avenue, along the buildings of the Exposition, it would be at the level of the second floor.

"Independently of the advantages which would result from this course, in respect to the enlargement of the different parts of the Exposition, the contractor places the combined capacity of the two systems at 50 000 or 60 000 passengers per hour. He says, besides, that the platform would be an attraction for the visitors to the Exposition.

"The tariff agreed upon is: Railroad, one and only class, 0.25 franc (5 cents); platform, one and only class, 0.50 franc (10 cents).

"The rent to go toward the receipts of the Exposition would be 1 centime (0.2 cent) per person carried by the railroad or by the platform.

"The railroad would be finished at the date fixed by the specifications, and the platform on March 1st, 1900."

"The Minister of Commerce and Industry is about to sign a concession for an electric railroad for the transportation of passengers in the interior of the Exposition of 1900. Among the five projects which have been submitted to the Commission having charge of the transportation department, the preference has been given to that of Mr. Mocomble. It is certainly the most original, and it also has the greatest transportation capacity.

"We will sum up briefly what the project, which has been worked up with the technical collaboration of Mr. Maréchal, Engineer of the Ponts et Chaussées, comprises: *First*, an electric railroad taking the current from a lateral rail, and moving in one direction on a course which forms, as is well known, a closed circuit; and, *second*, a moving platform at two rates of speed, and moving in an opposite direction from the railroad.

"This platform will carry a stationary sidewalk from which access can be had to the first platform, moving at a rate of 5 km. per hour; from this one can pass to the second platform, moving at double that rate of speed.

"The authors of the project attribute to the two combined methods of transportation a capacity of 50 000 to 60 000 passengers per hour.

"Conforming to one of the articles of the convention approved by the Minister of Commerce, a joint stock company will be formed in place of the holder of the concession. This company will take the name: 'Electrical Transportation Company of the Exposition,' and must proceed within six months with a trial of the moving sidewalk system adopted, on a closed circuit of 300 to 350 m. (984 to 1 148 ft.)."

The Metropolitan Underground Electric Railway.

The routes of this railway, under its rights granted by the Municipal Government of Paris, are shown in heavy solid lines on the General Map of Paris (Plate VI). Much of this work is now under way. It is not expected that the entire system will be completed before the opening of the Exposition, but those lines leading from the railway stations to the Exposition, and connecting the stations, will no doubt be ready for operation at that time. The whole system comprises from 35 to 40 miles.

The construction requires the relocation and reconstruction of sewers, water and gas pipes, electric conduits, etc. No work can be done in open cut; all must be in tunnel. This makes the work very expensive and very slow. The work is not generally very deep below the street surface. The material is good for excavation and handling, and is removed as fast as brought up from the shafts. Much of the work of carting it away is done at night. The plans generally contemplate two single-track tunnels of small dimensions. The single-car system will be used, and the traction plan will be by trolley, like the Boston subway. There is no connection with the railways, nor can there be any. The gauge is intentionally made different, and also the levels and other features. The fear that the railways might at some time get authority to run their cars into the city led the City Government to take steps at the outset to prevent it.

Paris depends largely upon its octroi duties for its income. Although its debts are large, its wealth in property is immense, and is said to be equal to that of some of the smaller kingdoms of the continent, like that of Portugal and Denmark.

The convenience of the people is a lesser consideration than the financial interests of the city. It is one of the principles of the Paris Government to keep the people within the area of the city proper. The octroi plan accomplishes this purpose; consequently, the districts

outside of the old line of fortifications do not build up to any great extent and there is, therefore, no great suburban business upon the railroads.

The details of the Metropolitan Railway plans have not been received by the author in time for this paper.

The Enlargement of the Gare de Lyons, of the Paris, Lyons and Mediterranean Railway.

The station is now being entirely remodeled at very great expense, said to reach $8 000 000. The new train-shed is completed, the work having been begun about two years ago. The plans for the main structure have just been approved, and work upon it will soon be begun. It will be very extensive and will be, when completed, the finest and most commodious station in Paris.

Exhibits.

The following exhibits accompanied the description of the approaches and transportation facilities of the Paris Exposition of 1900, and are filed in the Library of the Society.

Exhibit A. —Plan Général. Exposition Universelle de 1900.
" B. —Vue d'Ensemble de l'Exposition de 1900. (Reproduced as Plate V.)
" C. —Plan de l'Exposition Universelle de 1900. (Reproduced as Fig. 1.)
" D. —Outline Plan of Paris. (Reproduced as Plate VI.)
" E. —Elevation et Coupe Transversale du Pont Axexandre III. (Reproduced as Plate VII.)
" F. —Two Pamphlets, "Notes sur la Construction du Pont Alexandre III," par M. M. Résal et Alby. Nos. 6, 27 and 39 of the *Annales des Ponts et Chausées*. Including 11 plates, of which two are reproduced as Figs. 2 and 3.
" F, 1.—Ancien Pont Suspendu des Invalides.
" F, 2.—Construction du Pont Alexandre III. Elevation and sections.
" F, 3.—Plan du Pont Alexandre III (reproduced as Fig. 4).
" F, 4.—Détails des Caissons et des Échafaudages pour la Construction des Culées.
" F, 5.—Vue de la Caisson de la Rive Droite à la fin du Montage.
" F, 6.—Montage du Caisson de la Rive Droite.
" F, 7.—Graphiques de la Déscent des Caissons.
" F, 8.—Details of Masonry Abutments.
" F, 9.—Façade Latérale de la Culée.
" G, 1.—Prolongement de la Ligne d'Orléans au Quai d'Orsay. Plan Général.
" G, 2.—Profil en Long.
" G, 3.—Profils en Travers Principaux.
" G, 4.—Gare du Quai d'Orsay. (Reproduced as Fig. 6.)
" G, 5.—Gare du Quai d'Orsay. Plan au niveau du rez-de-chaussée.
" G, 6.—Gare du Quai d'Orsay. Plan au niveau des voies. (Reproduced as Plate VIII.)
" H, 1.—Chemin de Fer de l'Ouest, Ligne de Courcelles-Centure à Passy, et au Champ de Mars. Plan général.
" H, 2.—Profil en long.
" H, 3.—Encorbellments en béton armé, Systéme Hennebique. (Reproduced as Fig. 7.)
" I. —Pamphlet, "Notice sur la Construction du Pont Mirabeau. With two photographs.
" J. —"Le Prolongement de la Ligne de Sceaux vers l'intérieur de Paris." Par A. Brière et Henri de la Brosse. Paris, 1895. Vol. 1, Text; Vol. 2, Plates.

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TRANSACTIONS.

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No. 848.

THE ACCURACY AND DURABILITY OF WATER METERS.

By JOHN W. HILL, M. Am. Soc. C. E.

PRESENTED FEBRUARY 1ST, 1899.

WITH DISCUSSION.

By the misguided suffrage of his fellow citizens, the author happens to be a Trustee of Water-Works of Wyoming, one of the suburbs of Cincinnati, and among other tasks which have fallen to him in this capacity has been an examination of water meters adapted for use in a small water-works.

The primary question to be discussed was the relative value of high and low-priced meters for water measurement in domestic service pipes; next, the precision of registry of the different kinds and makes of meters, with widely varying rates of delivery; and, finally, the loss of pressure by passing different quantities of water through the meters in a given length of time. The unit of time has been taken at one hour, and the delivery of the meter is stated in gallons.

Seventeen different meters were examined, as follows:

The Worthington meter, from Henry R. Worthington, New York City.

The Union Rotary and Columbia meters, from The Union Water Meter Company, Worcester, Mass.

The Crown, Empire and Nash meters, from the National Meter Company, New York City.

The Hersey and Hersey Disc meters, from the Hersey Manufacturing Company, Boston, Mass.

The Lambert (Bee) meter, from The Thomson Meter Company, Brooklyn, N. Y.

The Trident Disc meter, of which several were purchased by the Village Water-Works, 1896.

The Niagara meter, from The Buffalo Meter Company, Buffalo, N. Y.

The Pittsburg Disc meter (two styles), from the Pittsburg Meter Company, East Pittsburg, Pa.

The Standard Disc meter, from The Standard Meter Company, New York City.

Of this list, the Worthington, Union Rotary, Crown, Empire and Hersey are so-called piston meters; the Nash, Hersey Disc, Lambert, Trident, Niagara, Pittsburg and Standard are disc meters, and the Columbia a velocity or inferential meter.

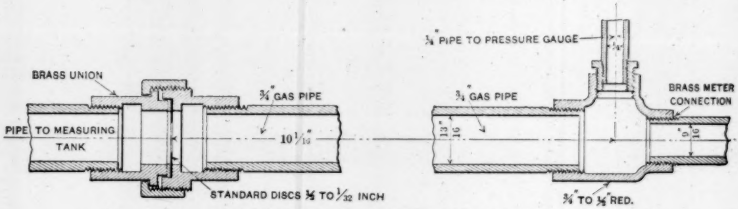
While the original object of this investigation was information for the guidance of the trustees of the village water-works in the selection of meters for local use, it has occurred to the author that the data may be of interest to others connected with the management of public water-works. Incidentally, it may be of some interest to the manufacturers of meters, and may possibly lead to the correction of defects which were manifested in some of the meters when tested on small streams.

The tests were made under the ordinary conditions of service to a residence, the ¾-in. service pipe being connected with the meters singly, and no flow through the pipe for domestic uses being permitted while a meter was under test.

The static pressure on the service pipe varied from 85 to 106 lbs., the main in the street in front of the building where the tests were conducted serving as a rising main to the reservoir during the day when the pumps were running, and as a supply main from the reservoir at night when the pumps were stopped, the pressure thus being highest during the day.

The distance from the 10-in. cast-iron main to the meter, measured along the ¾-in. iron supply pipe, was about 107 ft., and there were five ¾-in. bends. The supply was taken through an ordinary galvan-

ized iron pipe measuring $\frac{13}{16}$ in., internal diameter, with gas-pipe fittings. No effort was made to accommodate the meters in the arrangement of the pipe, the condition desired being as nearly as possible like that of a meter placed in the line of a domestic service pipe. The varying rates of discharge were obtained by placing in the short piece of $\frac{3}{4}$-in. pipe attached to the outlet connection of the meter, brass discs, 0.0265 in. thick, with central perforations ranging from $\frac{1}{2}$ in. to $\frac{7}{32}$ in. diameter. The discs were held against a thin rubber gasket in a brass union recessed as shown in Fig. 1. The flow of water to the meter was controlled by a lever stop-cock in the supply pipe below the pressure-gauge. During each test the stop-cock was set at full opening, and the flow of water through the meter controlled entirely by the standard orifice in the discharge pipe. Before any test was started, the meter was run for some time at full opening to work



DISCHARGE PIPE AND STANDARD ORIFICE.

FIG. 1.

out the air. The Worthington was the only meter provided with means for blowing the air out of the cylinders when it is started.

The apparatus used for the measurement of the discharge and the indication of the pressures in the inlet and outlet pipes of the meter is shown in Fig. 2. The cask on the platform scale had a capacity of 7 cu. ft., and the scale beam was graduated to half pounds. By hanging the sliding poise on the saw teeth of the beam, weights to quarter pounds could be noted.

The results of the tests of the meters at different rates of delivery are given in Table No. 1, first, for each of the piston meters; next, for the disc meters, and, finally, for the velocity or inferential meter. In all cases the meter was brought to the zero point of the 1-cu. ft. dial, the tare of the measuring cask taken, and from 1 to 6 cu. ft. of water drawn through the meter and caught in the cask. The net

contents of the cask were reduced to cubic feet for observed temperature of water, and the discharge, as noted by tank measurement, divided by the difference of meter reading, *i. e.*, by the discharge as indicated by the meter.

TABLE No. 1.

Piston Meters.

Résumé of Test of $\frac{5}{8}$-In. Worthington Meter, No. 100 020. October 20th and 21st, 1898.

Order of Test, 15.

Number of tests.	Diameter of orifice, in inches.	Pressure on supply pipe, in pounds.	Discharge, in gallons per hour.	Cubic feet, by meter.	Cubic feet, by tank.	Ratio of tank to meter.	Temperature of water, degrees.
3.....	$\frac{1}{2}$	29.66	828.27	5.000	5.0443	1.0089	56
3.....	$\frac{3}{4}$	65.00	531.30	5.000	5.0443	1.0089	56
4.....	$\frac{7}{8}$	92.47	172.25	3.000	2.9655	0.9885	57
6.....	$\frac{1}{16}$	98.86	45.34	2.016	2.0020	0.9933	58
2.....	$\frac{1}{32}$	87.77	10.82	1.000	0.9064	0.9064	58

Résumé of Test of $\frac{5}{8}$-In. Crown Meter, No. 187 595. October 7th and 10th, 1898.

Order of Test, 9.

4.....	$\frac{1}{2}$	32.75	872.85	5.000	5.2829	1.0566	56
3.....	$\frac{3}{4}$	69.66	546.31	4.333	4.5025	1.0392	58
2.....	$\frac{7}{8}$	92.95	172.68	3.000	3.1807	1.0602	60
3.....	$\frac{1}{16}$	99.76	45.66	3.000	3.2302	1.0767	60
3.....	$\frac{1}{32}$	93.62	11.18	1.000	1.1044	1.1044	61

Résumé of Test of $\frac{3}{4}$-In. Hersey Meter, No. 57 617. October 10th, 1898.

Order of Test, 10.

3.....	$\frac{1}{2}$	21.33	913.52	5.000	5.1889	1.0378	56
3.....	$\frac{3}{4}$	68.66	559.95	5.000	5.1701	1.0340	56
4.....	$\frac{7}{8}$	92.81	172.58	3.000	2.9841	0.9947	60
.....	$\frac{1}{16}$	Meter refused to work with this orifice.					

Résumé of Test of $\frac{5}{8}$-In. Empire Meter, No. 191 159. October 1st and 3d, 1898.

Order of Test, 1.

6.....	$\frac{1}{2}$	23.33	746.60	6.017	6.0523	1.0058	64
4.....	$\frac{3}{4}$	58.50	508.90	5.750	5.7665	1.0028	65
5.....	$\frac{7}{8}$	87.28	167.44	4.800	4.7902	0.9977	64
5.....	$\frac{1}{16}$	88.95	43.07	2.800	2.7845	0.9943	69
3.....	$\frac{1}{32}$	83.90	10.59	1.333	1.3460	1.0097	68

Résumé of Test of $\frac{5}{8}$-In. Union Rotary Meter, No. 67 225. October 18th, 1898.

Order of Test, 13.

3.....	$\frac{1}{2}$	20.00	940.94	5.000	4.9686	0.9967	58
3.....	$\frac{3}{4}$	71.00	581.90	5.000	4.9703	0.9942	58
3.....	$\frac{7}{8}$	98.70	178.06	3.000	2.9355	0.9786	59
3.....	$\frac{1}{16}$	102.29	46.23	2.000	1.9679	0.9840	59
4.....	$\frac{1}{32}$	88.72	10.88	1.000	1.1953	1.1953	61

TABLE No. 1—(*Continued*).

DISC METERS.

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. NASH METER, No. 191 160. OCTOBER 3D, 1898.

Order of Test, 2.

No. of tests.	Diameter of orifice, in inches.	Pressure on supply pipe, in pounds.	Discharge, in gallons per hour.	Cubic feet, by meter.	Cubic feet, by tank.	Ratio of tank to meter.	Temperature of water, degrees.
3.....	$\frac{1}{2}$	22.83	933.14	5.333	5.3603	1.0050	57
3.....	$\frac{1}{4}$	70.00	573.79	5.000	5.0000	1.0000	58
3.....	$\frac{1}{8}$	92.78	172.70	4.000	4.0300	1.0076	60
3.....	$\frac{1}{16}$	95.92	44.77	3.000	3.0191	1.0063	61
3.....	$\frac{1}{32}$	94.80	11.24	1.667	1.6690	1.0004	64

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. HERSEY METER, No. 68 629. OCTOBER 4TH AND 5TH, 1898.

Order of Test, 4.

3.....	$\frac{1}{2}$	33.33	868.87	5.000	5.0472	1.0094	57
2.....	$\frac{1}{4}$	74.00	556.68	5.000	5.0126	1.0025	57
5.....	$\frac{1}{8}$	92.43	172.11	3.000	2.9614	0.9871	59
3.....	$\frac{1}{16}$	94.71	44.50	3.000	2.9800	0.9933	65
2.....	$\frac{1}{32}$	90.22	10.98	2.000	1.9943	0.9972	65

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. TRIDENT METER, No. 23 178. OCTOBER 6TH, 1898.

Order of Test, 5.

3.....	$\frac{1}{2}$	25.00	926.78	5.000	5.0019	1.0004	57
3.....	$\frac{1}{4}$	66.67	556.23	5.000	4.9857	0.9971	57
3.....	$\frac{1}{8}$	93.76	173.52	3.000	2.9813	0.9938	59
3.....	$\frac{1}{16}$	95.09	44.57	2.000	2.0055	1.0027	60
3.....	$\frac{1}{32}$	87.52	10.80	1.333	1.3910	1.0446	64

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. TRIDENT METER, No. 23 179. OCTOBER 6TH, 1898.

Order of Test, 6.

2.....	$\frac{1}{2}$	23.00	900.72	5.000	5.0006	1.0001	57
2.....	$\frac{1}{8}$	89.10	169.19	3.000	3.0000	1.0000	59
2.....	$\frac{1}{32}$	90.55	10.99	1.000	1.2600	1.2600	65

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. TRIDENT METER, No. 23 180. OCTOBER 6TH AND 7TH, 1898.

Order of Test, 7.

2.....	$\frac{1}{2}$	24.00	873.18	5.000	5.0893	1.0178	58
2.....	$\frac{1}{8}$	87.87	167.95	3.000	3.0403	1.0134	60
2.....	$\frac{1}{32}$	89.77	10.95	1.000	1.1633	1.1633	60

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. TRIDENT METER, No. 23 176. OCTOBER 7TH, 1898.

Order of Test, 8.

3.....	$\frac{1}{2}$	24.67	915.81	5.000	5.0447	1.0089	57
2.....	$\frac{1}{8}$	94.85	174.50	3.000	3.0199	1.0066	59
3.....	$\frac{1}{32}$	90.41	10.99	0.833	1.8749	2.2921	62

TABLE No. 1—(*Concluded*).

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. PITTSBURG METER, No. 8 011. OCTOBER 3D AND 4TH, 1898.

Order of Test, 3.

Number of tests.	Diameter of orifice, in inches.	Pressure on supply pipe, in pounds.	Discharge, in gallons per hour.	Cubic feet, by meter.	Cubic feet, by tank.	Ratio of tank to meter.	Temp. of water, degrees.
2.....	$\frac{1}{2}$	23.00	896.92	5.000	5.0293	1.0060	59
2.....	$\frac{1}{4}$	65.75	548.74	5.000	5.0333	1.0067	59
2.....	$\frac{1}{8}$	96.04	175.71	3.000	3.0244	1.0081	60
2.....	$\frac{1}{16}$	95.40	44.65	3.000	3.0768	1.0255	62
3.....	$\frac{1}{32}$	93.09	11.15	1.667	1.8779	1.1200	64

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. PITTSBURG METER, No. 10 798. OCTOBER 10TH AND 11TH, 1898.

Order of Test, 11.

3.....	$\frac{1}{2}$	20.00	907.36	5.000	4.9863	0.9973	58
2.....	$\frac{1}{4}$	65.00	557.06	5.000	4.9649	0.9930	58
3.....	$\frac{1}{8}$	90.63	170.71	3.000	2.9520	0.9842	59
3.....	$\frac{1}{16}$	102.66	46.31	2.000	2.0351	1.0176	61
2.....	$\frac{1}{32}$	94.32	11.22	1.000	1.3278	1.3278	60

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. NIAGARA METER, No. 18 901. OCTOBER 17TH, 1898.

Order of Test, 12.

3.....	$\frac{1}{2}$	19.66	894.93	5.000	4.9843	0.9968	60
3.....	$\frac{1}{4}$	64.00	548.71	5.000	4.9313	0.9863	61
3.....	$\frac{1}{8}$	88.63	168.95	3.000	2.9258	0.9753	62
3.....	$\frac{1}{16}$	90.34	43.44	2.000	1.9792	0.9896	62
3.....	$\frac{1}{32}$	101.23	11.62	1.333	1.6048	1.2062	63

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. STANDARD METER, No..... OCTOBER 2D, 1898.

Order of Test, 16.

4.....	$\frac{1}{2}$	20.00	938.18	5.000	4.9120	0.9824	56
3.....	$\frac{1}{4}$	69.66	571.70	5.000	4.8973	0.9795	56
3.....	$\frac{1}{8}$	95.76	175.23	3.000	2.9459	0.9820	56
4.....	$\frac{1}{16}$	99.70	45.65	2.000	2.1535	1.0768	58
	$\frac{1}{32}$	Meter refused to work with this orifice.					

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. LAMBERT METER, No. 121 795. OCTOBER 24TH, 1898.

Order of Test, 17.

4.....	$\frac{1}{2}$	24.50	911.27	5.000	5.0423	1.0085	56
3.....	$\frac{1}{4}$	68.92	560.57	5.000	5.0443	1.0089	56
3.....	$\frac{1}{8}$	93.15	173.03	3.000	3.0394	1.0131	56
1.....	$\frac{1}{16}$	97.68	45.17	1.000	1.2110	1.2110	56

Upon next test with $\frac{1}{16}$-in. orifice the meter stuck.
The gear-train probably binds at top of case.

INFERENTIAL METER.

RÉSUMÉ OF TEST OF $\frac{5}{8}$-IN. COLUMBIA METER, No. 67 139. OCTOBER 18TH AND 19TH, 1898.

Order of Test, 14.

3.....	$\frac{1}{2}$	40.00	762.43	5.000	5.0397	1.0079	58
3.....	$\frac{1}{4}$	67.00	511.99	5.000	5.0638	1.0127	58
3.....	$\frac{1}{8}$	95.10	172.93	3.000	3.0720	1.0240	58
6.....	$\frac{1}{16}$	98.76	45.43	2.333	2.2165	0.9508	58
4.....	$\frac{1}{32}$	92.77	11.13	2.675	0.7947	0.2968	58

Considering the accuracy of small water meters, the author was surprised at the trifling variation in errors of registry for rates of discharge, varying in some examples as 90 to 1. Thus, the Empire meter, during 23 tests, with a range of delivery from 75 to 1, varied from a minus error of 0.87% to a plus error of 0.23%, with an average error of — 0.18 per cent. The Nash meter, with a discharge varying from 83 to 1, exhibited during the 15 tests a maximum error of — 0.75%, and a minimum error of — 0.04%, with an average error of registry of 0.38 per cent. These were new meters. The Crown meter had passed 3 500 cu. ft. of water prior to the test, but, under the usual conditions of meter service, this should have had no apparent effect on its accuracy. This meter showed a maximum error of — 9.45%, and a least error of — 3.77%, with an average error for 15 tests of — 6.28 per cent. The Hersey disc meter, for 15 tests, with a range of delivery from 80 to 1, showed an average of + 0.22%, a maximum error of + 1.31%, and a minimum error of — 0.25 per cent. The Trident meter, No. 23 178, for 15 tests, gave a maximum error of — 4.27%, a minimum error of — 0.04%, and an average error of — 0.73 per cent. Table No. 2 shows the errors of all the meters tested.

TABLE No. 2.—Comparison of Errors of Registry.

Order of test.	Meter.	Number of tests.	Error.		
			Maximum.	Minimum.	Average.
	Piston Meters.				
15	Worthington	18	+10.32	+0.67	+ 2.08
9	Crown	15	— 9.45	—3.77	— 6.28
10	Hersey	10	— 3.64	+0.53	— 2.13
1	Empire	23	— 0.86	+0.23	— 0.18
13	Union Rotary	16	—16.34	+0.33	— 2.32
	Disc Meters.				
2	Nash	15	— 0.75	—0.04	— 0.38
4	Hersey	15	+ 1.31	—0.25	+ 0.22
5	Trident, No. 23 178	15	— 4.27	—0.04	— 0.73
6	Trident, No. 23 179	6	—20.63	—0.01	— 6.88
7	Trident, No. 23 180	6	—14.08	—1.32	— 5.70
8	Trident, No. 23 176	8	—56.37	—0.66	—19.30
3	Pittsburg, No. 8 011	11	—10.71	—0.60	— 3.05
11	Pittsburg, No. 10 798	13	—24.68	+0.27	— 4.77
12	Niagara	15	—17.10	+0.32	— 2.36
16	Standard	14	— 7.13	+1.79	— 0.35
17	Lambert	11	—17.42	—0.84	— 5.11
	Inferential Meter.				
14	Columbia	19	+66.31	—0.78	+13.42

An error is stated as plus when the registry of the meter was greater than the actual quantity of water delivered, and as minus when the quantity of water delivered exceeded the registry of the meter.

Certain meters, while quite accurate with ordinary rates of draught, were incapable of measuring the smaller discharges, and these smaller discharges more nearly resemble the leakage in domestic plumbing, which, with the meter system of water measurement, the average householder will be prompt to correct, in the interest of domestic economy.

This inquiry was undertaken altogether from a practical point of view, and the author has no doubt that certain meters which did not show quite as good results as other meters may be just as reliable instruments for the ordinary measurement of water supplied through service pipes.

Apart from any refinements of water measurement which may be shown by these and former tests of meters, one cannot neglect the important fact that, however precise the instrument may be under test, it must be constructed on such principles and in such a manner as will guarantee its durability and continued proper working through years of time; and it should be very gratifying to all interested in the use of meters that those meters, which in these tests have shown the greatest precision of measurement, also have demonstrated their capacity to operate for a generation or longer in daily service, and still be in a condition to satisfactorily approximate the quantity of water which is passed through them.

As a general proposition, only two or three of the meters fell so low in the author's esteem as to make them doubtful devices for the measurement of water; but any meter represented in these tests (if not damaged by use) is superior to the prevailing mode of charging for water by a survey of the premises. So long as many consumers of water from the public mains are wasting as much as they use, while others are using less than half of what they pay for, any water meter that will work with all streams, and indicate the actual quantity of water passed within 4 or 5%, will meet all present requirements. When water meters, as they should be, are used to measure every gallon of water taken for consumption from the public mains, then the refinements of meter measurement can be safely discussed.

TABLE No. 3.—Comparison of Wyoming and Boston Meter Tests.

Wyoming Tests, 1898.				Boston Tests, 1887–88.			
Diameter of orifice.	Pressure, in pounds.	Ratio of tank to meter.	Discharge, gallons per hour.	Diameter of orifice.	Pressure, in pounds.	Ratio of tank to meter.	Discharge, gallons per hour.
⅝-in. Crown, No. 187 595.				¾-in. Crown, No. 45 073.			
½-in.	32.75	1.0566	872.85	½-in.	21	0.9902	1 032.24
¼-in.	69.66	1.0392	546.31	¼-in.	41	0.9902	448.80
⅛-in.	92.95	1.0602	172.68	0.15-in.	45.5	1.0021	184.01
1/16-in.	99.76	1.0767	45.66	0.05-in.	46	1.0366	23.34
1/32-in.	93.62	1.1044	11.18	0.03-in.	46	1.0552	8.53
⅝-in. Empire, No. 191 159.				¾-in. Empire, No. 45 075.			
½-in.	23.33	1.0058	746.60	½-in.	18.5	1.0112	1 081.61
¼-in.	58.50	1.0028	503.90	¼-in.	41	0.9971	439.82
⅛-in.	87.28	0.9977	167.44	0.15-in.	45	0.9873	184.01
1/16-in.	88.95	0.9943	43.07	0.05-in.	46	1.0941	21.09
1/32-in.	83.90	1.0087	10.59	0.03-in.	46	1.4791	8.53
¾-in. Hersey Rotary, No. 57 617.				¾-in. Hersey Rotary, No. 448.			
½-in.	21.33	1.0378	913.52	½-in.	23	0.9873	973.90
¼-in.	68.66	1.0340	559.95	¼-in.	40	0.9872	453.29
⅛-in.	92.81	0.9947	172.58	0.15-in.	45	0.9707	188.49
⅝-in. Union, No. 67 225.				¾-in. Union, No. 23 816.			
½-in.	20.00	0.9967	940.94	½-in.	17	1.0190	1 095.07
¼-in.	71.00	0.9942	581.90	¼-in.	40	1.0051	435.34
⅛-in.	98.70	0.9786	178.06	0.15-in.	44	1.0149	175.03
1/16-in.	102.29	0.9840	46.23	0.05-in.	46	8.8471	22.89
1/32-in.	88.72	1.1953	10.88	0.03-in.	46.5	91.9166	8.53
⅝-in. Worthington, No. 100 020.				¾-in. Worthington, No. 5 990.			
½-in.	29.66	1.0089	828.27	½-in.	21	0.9974	1 050.19
¼-in.	65.00	1.0089	531.30	¼-in.	42	0.9810	444.31
⅛-in.	92.47	0.9885	172.25	0.15-in.	47	0.9847	192.98
1/16-in.	98.36	0.9933	45.34	0.05-in.	46	0.8848	22.89
1/32-in.	87.77	0.9064	10.82	0.03-in.	47	0.9466	8.98

The perfect meter is in the future, and doubtless will contain the principle laid down by John Thomson, M. Am. Soc. C. E.;* but, meanwhile, no one is justified in rejecting the modern displacement meter, as a device for the measurement of water, upon the theory that it is not sufficiently accurate for all practical purposes.

Water meters are not made, and should not be rated, like "Jurgensen" watches, and errors much in excess of those mentioned are admissible in practice without indicating that the water meter is a faulty machine for its purpose. In his excellent paper on water meters, Mr. Thomson states that errors of 5 to 7% should not condemn a water meter, and the author can accept this approach to accuracy of measurement provided he is quite sure that the variations of the meter from the true discharge do not exceed the limits laid down. In these and other tests of water meters which the author

*"A Memoir on Water Meters," *Transactions*, Am. Soc. C. E., Vol. xxv, p. 65.

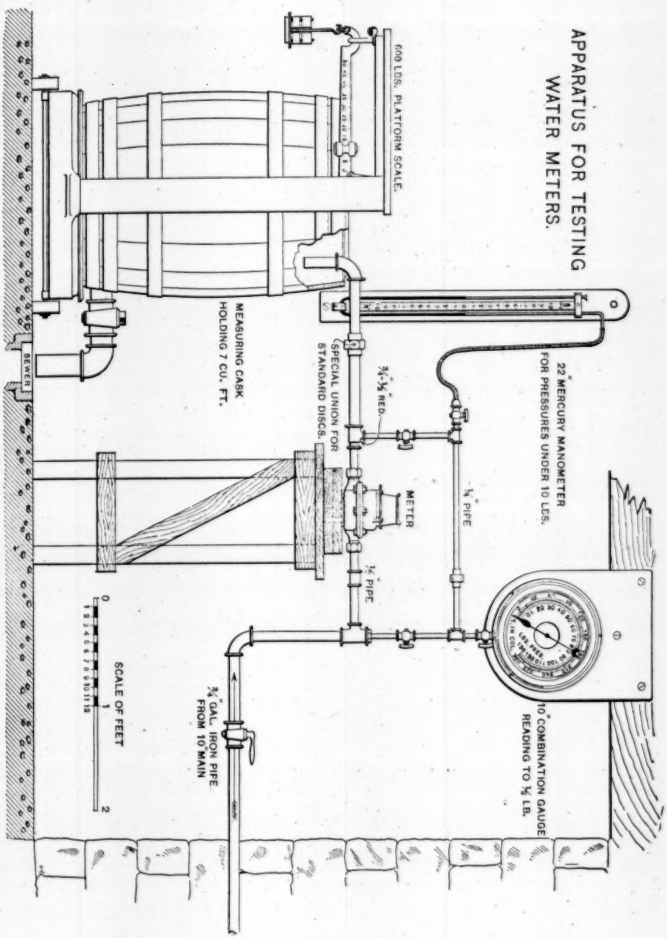

APPARATUS FOR TESTING WATER METERS.
600 LBS. PLATFORM SCALE.
22" MERCURY MANOMETER FOR PRESSURES UNDER 10 LBS.
MEASURING CASK HOLDING 7 CU. FT.
¾"-½" RED.
SPECIAL UNION FOR STANDARD DISCS.
METER
¼" PIPE
¾" PIPE
10" COMBINATION GAUGE READING TO ½ LB.
¾" GAL. IRON PIPE FROM 10" MAIN
SCALE OF FEET
0
1
2
BENCH

FIG. 2.

has made, the errors sometimes go considerably beyond 5 or 7%, and thus raise a doubt as to the general accuracy of these devices for the measurement of the widely varying streams which flow through domestic service pipes.

Perhaps the most elaborate tests of water meters ever made were conducted under the auspices of the Boston Water Board, during 1887-88, and, curiously enough, of the 26 meters tested there, only five or six, within the author's knowledge, are now offered in the market. As a matter of interest, the five meters tested, both at Boston and Wyoming, are brought together for the nearest concordant discharges in Table No. 3.

The errors of registry for these meters by these tests were as follows:

TABLE No. 4—Errors of Registry.

Meter.	Wyoming Test.			Boston Test.		
	Maximum.	Minimum.	Average.	Maximum.	Minimum.	Average.
Crown.....	— 9.45	— 3.77	— 6.28	— 5.23	+ 0.99	— 1.40
Empire.....	— 0.86	+ 0.23	— 0.18	+ 32.39	+ 1.29	— 8.10
Hersey.....	— 3.64	+ 0.53	— 2.13	+ 3.02	+ 1.29	+ 1.87
Union.....	— 16.34	+ 0.33	— 2.33	+ 98.91	— 0.51	— 38.31
Worthington....	+ 10.32	+ 0.67	+ 2.08	+ 13.02	— 0.26	+ 4.48

Loss of Head and Reduced Capacity of Service Pipes by Meters.

During the time the meters were being tested by the author for accuracy at different rates of discharge, the pressures were taken only on the inlet side of the meter. Subsequently the pipes were arranged, as shown in Fig. 2, to take pressures on both sides of the meter. During these tests, the standard orifices were used in the discharge pipe, and pressures under 10 lbs. were taken on the mercury piezometer shown in Fig. 2.

A careful review of Table No. 5, and of Table No. 7, giving the reduced discharges with meters connected in the test apparatus, indicates that the disc meters consumed less of the static head on the service pipe, and therefore realized a larger percentage of reduced capacity than the piston or inferential meters. If the disc meter can be shown to be as durable in service, and in other respects equal to the piston and inferential meters, it is entitled to first place in the types of water meters. The author's tests were not made to show the

durability of water meters, but, from authentic data placed in his hands for professional purposes by a city water department, he is led to the conclusion that the present disc meter (if not so in all examples at present), can be made as durable as the other types of meters; and if this opinion is well founded, it must become the standard of water meters for domestic uses.

TABLE No. 5—Loss of Head by Meter.

Meter.	Diameter of orifice, in inches.	Pressure, pounds, inlet.	Pressure, pounds, outlet.	Difference, loss of head.	Percentage, loss of head.
Piston Meters.					
⅝-in. Worthington, No. 100 020...	⅛	29.00	4.69	24.30	83.81
	¼	60.50	50.50	10.00	16.53
	⅜	85.00	84.00	1.00	1.18
⅝-in. Crown, No. 187 595.....	⅛	29.75	4.79	24.96	83.90
	¼	68.50	57.00	11.50	16.79
	⅜	91.00	90.00	1.00	1.10
¾-in. Hersey, No. 57 617.....	⅛	23.00	5.89	17.11	74.38
	¼	67.50	59.50	8.00	11.85
	⅜	91.00	90.00	1.00	1.10
⅝ in. Empire, No. 191 159.....	⅛	27.50	4.17	23.33	84.84
	¼	68.25	58.25	10.00	14.65
	⅜	91.00	90.25	0.75	0.82
⅝-in. Union Rotary, No. 67 225...	⅛	17.50	4.88	12.62	72.11
	¼	61.00	56.00	5.00	8.20
	⅜	85.00	84.25	0.75	0.88
Disc Meters.					
⅝-in. Nash, No. 191 160.....	⅛	21.00	5.40	15.60	74.29
	¼	66.00	59.50	6.50	9.85
	⅜	90.75	90.00	0.75	0.83
⅝-in. Hersey, No. 68 629.....	⅛	31.50	4.82	26.68	84.70
	¼	69.00	57.50	11.50	16.66
	⅜	91.50	90.50	1.00	1.09
⅝-in. Trident, No. 23 176.....	⅛	22.50	4.98	17.52	77.87
	¼	67.00	59.00	8.00	11.94
	⅜	91.00	90.25	0.75	0.82
⅝-in. Pittsburg, No. 8 011.....	⅛	21.00	4.66	16.33	77.76
	¼	67.00	60.00	7.00	10.45
	⅜	91.50	90.75	0.75	0.82
⅝-in. Pittsburg, No. 10 798.....	⅛	20.00	5.28	14.72	73.60
	¼	67.00	60.50	6.50	9.70
	⅜	92.75	92.00	0.75	0.81
⅝-in. Niagara, No. 18 901.....	⅛	20.50	5.57	14.92	72.80
	¼	67.25	60.75	6.50	9.67
	⅜	91.75	91.25	0.50	0.54
⅝-in. Standard, No.	⅛	18.50	5.03	13.47	72.81
	¼	67.50	62.00	5.50	8.15
	⅜	91 75	90.75	1.00	1.09
⅝-in. Lambert, No. 121 795.....	⅛	21.50	5.03	16.47	76.59
	¼	61.50	54.00	7.50	12.19
	⅜	85.50	84.50	1.00	1.17
Inferential Meter.					
⅝-in. Columbia, No. 67 139.....	⅛	37.50	3.44	34.06	90.83
	¼	64.50	48.50	16.00	24.81
	⅜	85.50	83.00	2.50	2.92

Losses of head with $\frac{1}{16}$ and $\frac{3}{32}$-in. orifices not measurable by the gauges used.

It is not apparent to the author why a disc meter should be subject to more rapid wear than a piston meter, nor why it should be less

convenient or more expensive to repair. It may be true that the rotary piston meters, when leakage becomes so great as to demand repair, can be taken apart and the piston box ground down or reduced in depth until the vertical play of the piston is no greater than in a new meter; but the cost of doing this—at least with small meters—will be as much or more than the cost of replacing a worn-out disc. If the pistons of the piston meters wear sufficiently at the displacement surfaces to render the registry unreliable, then new pistons become necessary, the cost of which will be much greater than new discs for the disc meters.

The loss of head and reduced discharge of house taps by the introduction of water meters may in some instances be a serious drawback, especially when the consumption should at times quite equal the capacity of the service pipe. Upon the other hand, it may be urged that the small difference of time required to fill a water-closet tank, bath tub or other domestic convenience, can constitute no serious objection to the slower meters; but in the event of water being taken through meters for domestic fire protection, then the loss of pressure and capacity becomes a very serious objection. Such a condition in connection with meters fortunately does not often arise, but it is entitled to some consideration, and manufacturers should strive to construct their machines so as to consume as little as possible of the pressure in the service pipe, while the water is passing through the meter and moving the working parts.

After the tests for accuracy of registry with the different rates of discharge, the apparatus shown in Fig. 2 was connected up with a piece of $\frac{5}{8}$-in. galvanized iron pipe, with the burrs at the ends filed out and all visible obstructions removed, with the results noted in Table No. 6:

TABLE No. 6.—TEST FOR REDUCED DISCHARGE OF APPARATUS WITH METER REMOVED.

No. of test.	Diameter of orifice, in inches.	Time, in minutes.	Pressure in supply pipe, in pounds.	Cubic feet.	Temperature of water, degrees.	Discharge, in gallons per hour.
1.....	$\frac{1}{2}$	2	15.5	4.2745	56	959.198
2.....	$\frac{1}{2}$	2	15.3	4.2664	56	957.380
3.....	$\frac{1}{2}$	2	15.5	4.2944	56	950.199
17.....	$\frac{1}{2}$	2.5	16.0	5.4052	56	970.341
18.....	$\frac{1}{2}$	2.5	16.0	5.4052	56	970.341
4.....	$\frac{1}{4}$	4	68.7	5.1406	56	576.775
5.....	$\frac{1}{4}$	4	68.8	5.1486	56	577.673
6.....	$\frac{1}{4}$	4	68.8	5.1486	56	577.673
7.....	$\frac{1}{8}$	8	94.7	3.1038	57	174.123
8.....	$\frac{1}{8}$	8	94.4	3.1038	57	174.123
9.....	$\frac{1}{8}$	8	95.0	3.1198	57	175.004

TABLE No. 7.—Reduced Discharge of Test Apparatus.

Meter.	Diameter of orifice, in inches.	Pressure on supply pipe, in pounds.	Gallons per hour, by meter.	Gallons per hour, by curve.	Percentage of reduced capacity.
Piston Meters.					
Worthington	½	29.66	828.27	1322.0	62.65
Crown	½	32.75	872.85	1392.0	62.70
Hersey	½	21.33	913.52	1120.0	81.56
Empire	½	23.33	746.60	1170.0	63.81
Union	½	20.00	940.94	1086.0	86.64
Worthington	¼	65.00	531.30	561.3	94.66
Crown	¼	69.66	546.31	581.0	94.03
Hersey	¼	68.66	559.95	576.8	97.08
Empire	¼	58.50	503.90	532.5	94.63
Union	¼	71.00	581.90	586.8	99.16
Worthington	⅛	92.47	172.25	172.6	99.77
Crown	⅛	92.95	172.68	173.1	99.74
Hersey	⅛	92.81	172.58	173.0	99.76
Empire	⅛	87.28	167.44	167.7	99.82
Union	⅛	98.70	178.06	178.3	99.84
Disc Meters.					
Nash	½	22.83	933.14	1158.0	80.58
Hersey	½	33.33	868.87	1403.0	61.93
Trident, No. 23 178	½	25.00	926.78	1215.0	76.28
Trident, No. 23 179	½	23.00	900.72	1163.0	77.45
Trident, No. 23 180	½	24.00	873.18	1188.0	73.50
Trident, No. 23 176	½	24.67	915.81	1205.0	76.00
Pittsburg, No. 8 011	½	23.00	896.92	1163.0	77.13
Pittsburg, No. 10 798	½	20.00	907.36	1086.0	83.55
Niagara	½	19.66	894.93	1076.0	83.17
Standard	½	20.00	938.18	1086.0	86.39
Lambert	½	24.50	911.27	1202.0	75.81
Nash	¼	70.00	573.79	582.5	98.50
Hersey	¼	74.00	556.68	599.0	92.93
Trident, No. 23 178	¼	66.67	556.23	568.2	97.89
Pittsburg, No. 8 011	¼	65.75	548.74	564.5	97.21
Pittsburg, No. 10 798	¼	65.00	557.06	561.3	99.24
Niagara	¼	64.00	548.71	556.9	98.53
Standard	¼	69.66	571.70	581.0	98.40
Lambert	¼	68.92	560.57	578.0	96.98
Nash	⅛	92.78	172.70	172.9	99.88
Hersey	⅛	92.43	172.11	172.6	99.72
Trident, No. 23 178	⅛	93.76	173.52	173.8	99.81
Trident, No. 23 179	⅛	89.10	169.19	169.5	99.82
Trident, No. 23 180	⅛	87.87	167.95	168.3	99.76
Trident, No. 23 176	⅛	94.85	174.50	174.8	99.80
Pittsburg, No. 8 011	⅛	96.04	175.71	176.0	99.84
Pittsburg, No. 10 798	⅛	90.63	170.71	170.9	99.86
Niagara	⅛	88.63	168.95	169.1	99.91
Standard	⅛	95.76	175.23	175.7	99.73
Lambert	⅛	93.15	173.03	173.3	99.84
Inferential Meter.					
Columbia	½	40.00	762.43	1536.6	49.62
Columbia	¼	67.00	511.99	569.8	89.85
Columbia	⅛	95.10	172.93	175.1	98.73

In these tests a piece of pipe of the same length as the average measurement from out to out of meter spuds was substituted for the meters for comparison of the discharge of the simple pipe arrangement with the discharge of the several meters. From the data thus

TABLE No. 8.—COMPARISON OF WYOMING AND HAMBURG METER TESTS.

No. of test.	Diameter of orifice, in inches.	Pressure, in pounds.	Loss of head, in pounds.	Ratio of tank to meter.	Discharge, gallons per hour.
		⅝-in. Lambert, No. 121 795 (Wyoming).			
223–226	½	24.50	18.75	1.0085	911.27
227–229	¼	68.92	8.40	1.0089	560.57
230–232	⅛	93.15	1.09	1.0131	173.03
233–234	1/16	97.68	Not measurable.	1.2110	45.17
	1/32	.....	" "	.....	.....
		¾-in. Lambert, No. 55 036 (Hamburg).			
15	....	44.1	7.676	1.000	834.34
14	....	44.1	2.843	1.000	511.23
12	....	44.1	0.426	1.000	194.18
8	....	44.1	Not measurable.	1.017	37.52
4	....	44.1	" "	1.053	7.66
		⅝-in. Nash, No. 191 160 (Wyoming).			
27–29	½	22.83	16.96	1.0050	983.14
30–32	¼	70.00	6.89	1.0000	573.79
24–26	⅛	92.78	0.77	1.0076	172.70
33–35	1/16	95.92	Not measurable.	1.0063	44.77
36–38	1/32	94.80	" "	1.0004	11.24
		¾-in. Nash, No. 133 225 (Hamburg).			
20	....	44.1	4.265	1.020	849.14
19	....	44.1	1.422	1.010	509.64
17	....	44.1	0.142	1.008	192.87
12	....	44.1	Not measurable.	1.000	50.73
6	....	44.1	" "	1.075	10.56
		⅝-in. Trident, No. 23 178 (Wyoming).			
63–65	½	25.00	19.47	1.0004	926.78
66–68	¼	66.67	7.96	0.9971	556.23
69–71	⅛	93.76	0.77	0.9938	173.52
72–74	1/16	95.09	Not measurable.	1.0027	44.57
75–78	1/32	87.52	" "	1.0446	10.80
		¾-in. Trident, No. 7 197*a* (Hamburg).			
12	....	44.1	8.103	1.000	1 170.40
11	....	44.1	1.706	1.000	524.44
10	....	44.1	0.284	0.996	201.58
5	....	44.1	Not measurable.	0.990	38.57
3	....	44.1	" "	1.010	11.36

obtained, curves were plotted for the ½-in., ¼-in. and ⅛-in. orifices, with the abscissas taken as the discharges in gallons per hour, and the ordinates as pressures in the service pipe. The curves were drawn on the theory that the discharge of a meter for a given orifice will vary as the square root of the pressure on the inlet side; and the actual discharge of a meter is compared with the discharge by the curve, for the given pressure on the inlet, to determine the loss of capacity by the resistances in the meter and its connections. The relation of the discharges by meters to the discharges by the plain pipe of the same approximate length as the separation of meter spuds, is given in Table No. 7.

A few years ago the water-works of Hamburg, under the direction of Herr O. Iben, conducted a series of tests upon three of the meters

represented in the Wyoming tests. The results of these tests, as furnished by a pamphlet issued by The Neptune Meter Company, of New York, and of the Wyoming tests, for the rates of discharge most nearly agreeing, are compared in Table No. 8.

The Trident and Nash meters, which came under Mr. Iben's observation, seem to have been not quite as accurate as those tested by the author, while the Lambert meter, tested abroad, gave very much better results in detail and as an average.

TABLE No. 9.—AVERAGE ERRORS OF REGISTRY.

Meter.	Wyoming test.	Hamburg test.
Trident	−0.73	+0.80
Nash	−0.38	−2.06
Lambert (Bee)	−5.11	−1.34

Three points are considered by Mr. Iben as essentials of a good water meter: (1) Sensitiveness or accuracy of registry at different rates of discharge; (2) capacity for a given size of meter, and (3) cost. To these should be added durability, without which any meter, however excellent it otherwise may be, will be a commercial failure. The meters in use in Germany are generally of the Siemens or inferential type, but much more complicated than the Columbia meter, represented in the Wyoming tests, which, excepting with the small streams of about 11 galls. per hour, gave very accurate results, but with a considerable reduction of pressure and capacity at the service pipe.

The use of water meters is much more general in Germany than in the United States. From a compilation of statistics of the waterworks of one hundred and twenty-one cities in Germany, including two in Holland, one in Denmark and one in Sweden,* the following information with reference to metered and unmetered water is taken.

In one hundred and eleven cities the total consumption of water for 1895 is stated as 445 158 552 cu. m. (117 611 000 000 galls.), of which quantity 220 158 552 cu. m. are metered, and 222 604 203 cu. m. are unmetered. It is not stated how the water is accounted for in the other ten cities consuming 31 542 777 cu. m. per year. Of these ten cities all excepting Zurich, Magdeburg and Leipsic are of small individual population. So far as the statistics indicate, about half the

* Statistische Zusammenstellung der Betriebs-Ergebnisse von Wasserwerken, Munich, 1895.

TABLE No. 10.—Percentage Losses of Head by Meters.

Wyoming.			Boston.		
Meter.	Gallons per hour.	Per cent. loss of head.	Meter.	Gallons per hour.	Per cent. loss of head.
⅝-in. Crown.....	872.8	83.90	¾-in. Crown.....	1032.2	40.48
½-in. connections .	546.2	16.79	½-in. connections .	448.8	7.32
	172.7	1.10		184.0	3.30
⅝-in. Worthington	828.3	83.81	¾-in. Worthington	1050.2	38.10
½-in. connections .	531.3	16.53	½-in. connections .	444.3	7.14
	172.2	1.18		193.0	4.25
⅝-in. Union	940.9	72.11	¾-in. Union	1095.1	17.65
½-in. connections .	581.9	8.20	½-in. connections .	435.3	4.17
	178.0	0.88		175.0	2.27
⅝-in. Hersey	913.5	74.38	¾-in Hersey	973.9	52.17
½-in. connections .	559.9	11.85	½-in. connections .	453.3	6.25
	172.6	1.10		188.5	2.22
⅝-in. Empire	746.6	84.84	¾-in. Empire.....	1081.6	27.08
½-in. connections .	503.9	14.65	½-in. connections .	439.8	2.44
	167.4	0.82		184.0	4.44
Wyoming.			Hamburg.		
⅝-in. Lambert.....	911.3	76.59	¾-in. Lambert.....	884.3	17.41
½-in. connections .	560.6	12.19	½-in. connections .	511.2	6.45
	173.0	1.17		194.2	0.97
⅝-in. Trident.....	926.8	77.87	¾-in. Trident.....	1170.4	18.37
½-in. connections .	556.2	11.94	½-in. connections .	524.4	3.87
	173.5	0.82		201.6	0.64
⅝-in. Nash.....	933.1	74.29	¾-in. Nash.....	849.1	9.67
½-in. connections .	573.8	9.85	½-in. connections .	509.6	3.22
	172.7	0.83		192.9	0.32

total consumption of water in the cities of Germany is metered, and by a device which will not compare in accuracy of measurement with the displacement water meters made and used in the United States.

In the London water-works different practices are followed by the several companies.* The New River Company meters all water supplied to large manufacturers and railway companies, the water used for street sprinkling, and sometimes the water for sewer flushing, amounting in all to 22.76% of the whole quantity supplied. The West Middlesex Company does not regularly meter any part of its supply. The Southwark and Vauxhall Company meters 18.87% of all the water it furnishes. The Chelsea Company meters all water supplied to the modern office and flat buildings, representing 23.6% of the total supply. The Lambeth Company meters 11% of its total supply. The East London Company meters 19.8% of its total supply. The Grand Junction Company meters a very small part (about 1%) of the water supplied, and the Kent Works has meters on 16.43% of all its services. In some examples the water is metered to a district through the

*Report of Royal Commission on Metropolitan Water Supply, London, 1893.

Deacon waste detectors furnished with recording apparatus. The intermittent supply to tanks, by the older practice of furnishing water to houses in London, in many instances renders meters less necessary than in the wholly open or undefended services in cities of the United States.

For purposes of comparison the meters tested, both at Boston and Hamburg, and by the author, are brought together in Table No. 10. In comparing the losses of head in these tests, it should be noted that at Boston and Hamburg all were $\frac{3}{4}$-in. meters with $\frac{3}{4}$-in. connections, while the Wyoming meters (excepting the Hersey) were all $\frac{5}{8}$-in., with standard $\frac{1}{2}$-in. connections.

The $\frac{1}{2}$-in. connection is actually $\frac{9}{16}$ in. in diameter, while the $\frac{3}{4}$-in. connection is actually $\frac{13}{16}$ in. in diameter; and assuming the loss of head, *i. e.*, the resistance by meter, to vary nearly as V^2, then the losses of head by the author's tests are with few exceptions uniformly lower than the losses reported for the Boston and Hamburg tests. Considering only the meters used in the Boston and Wyoming tests, it is possible that the latter may contain better proportions and workmanship than meters from the same manufacturers ten years ago.

Durability of Water Meters.

By way of showing the steady reputation which the Worthington water meter has sustained through a long history of actual service in many water-works, the Wyoming tests of $\frac{5}{8}$-in. meter No. 100 020 are compared with the Boston test of $\frac{3}{4}$-in. meter No. 5 990, and the San Francisco test of $\frac{5}{8}$-in. meter No. 26 737.*

TABLE No. 11.—Comparison of Three Worthington Meters.

Meter.	Diameter, of orifice, in inches.	Pressure, in pounds.	Ratio of tank to meter.	Discharge, gallons per hour.
$\frac{5}{8}$-in. meter, No. 100 020, tested at Wyoming, 1898	$\frac{1}{2}$	29.66	1.0089	828.27
	$\frac{1}{4}$	65.00	1.0089	531.30
	$\frac{1}{8}$	92.47	0.9885	172.25
	$\frac{1}{16}$	98.36	0.9933	45.34
	$\frac{1}{32}$	87.77	0.9064	10.82
$\frac{3}{4}$-in. meter, No. 5 990, tested at Boston, 1887-88	$\frac{3}{4}$	21.00	0.9974	1 080.19
	"	42.00	0.9810	444.31
	0.15	47.00	0.9847	192.98
	0.05	46.00	0.8848	22.89
	0.03	47.00	0.9466	8.98
$\frac{5}{8}$-in. meter, No. 26 737, tested at San Francisco, 1884	.....	18.79	1.0138	742.20
	.....	23.13	0.9958	444.00
	.....	25.00	0.9815	194.00
	.....	25.30	0.9519	52.20
	.....	25.37	0.9200	10.80

* Ross E. Browne, "Water Meters." Van Nostrand Scientific Series, No. 81, p. 47.

The principle of the Worthington meter is such that it was a perfected machine in its original conception, and no improvements were possible, excepting as they related to materials of construction and proportion of parts. It was a characteristic of the late Henry R. Worthington, M. Am. Soc. C. E., that all his inventions were perfected when they took form in his great mind.

Upon comparing the errors of these three meters, widely separated in numbers and time of construction, it appears that the earlier meters were nearly as accurate as those of to-day.

TABLE No. 12.—ERRORS OF REGISTRY.

Meter.	Error.		
	Maximum.	Minimum.	Average.
Worthington, No. 100 020	+10.32	+0.67	+2.08
Worthington, No. 5 990	+13.02	−0.26	+4.48
Worthington, No. 26 737	+ 8.70	+0.42	+2.94

Considering the durability of water meters, the records of many water departments will show that some meters have been in service for many years and have given no serious trouble. How well these old meters are accounting for the water passing through them can be known only by removing them from the service pipes and testing them, or by providing facilities for testing them in the line of pipe. Very little information is forthcoming upon the accuracy of water meters after reasonable wear, but, so far as the author's experience has gone, this is a feature that deserves more serious consideration than seems to have been given to it thus far.

In order to obtain information on the wearing properties of water meters, a city water department took six $\frac{5}{8}$-in. meters from stock, and submitted them to the tests detailed in Table No. 13.

All these meters were obtained from well-known manufacturers, and those which passed the test successfully completed a performance quite the equivalent of 20 to 40 years in an ordinary domestic service pipe.

Meters "B," "C," "D," "E" and "F" were run beyond their rated capacity, and failure under these conditions was more likely to

TABLE No. 13.

Designation of meters.	Type of meter.	Days of service.	Cubic feet passed.	Average gallons per hour.	Remarks.
"A".....	Inferential	16	29 974	583.9	Failed to register on 16th day.
"B".....	Disc	3	11 260	1 169.9	Failed to register. Disc cracked on 3d day.
"C".....	Disc	41	204 218	1 552.5	Meter in serviceable condition at end of trial.
"D".....	Disc	25	111 362	1 388.4	Stem of disc ball broken on 25th day.
"E".....	Disc	40	202 276	1 576.2	Meter in serviceable condition at end of trial.
"F".....	Disc	45	200 000	1 385.3	Meter in serviceable condition at end of trial.

occur than if they had been operated for twice the length of time at half the stated rate of delivery. The performance of meters "C," "E" and "F" is marvelous, and should convince any water department about to adopt meters that a well-made meter is almost as durable as a stop-valve or piece of cast-iron pipe.

An examination of the records in detail of meters "C," "E" and "F" indicates a very uniform operation from day to day throughout the whole period of time, and considering the enormous number of pulsations of the discs (from 63 000 000 to 90 000 000) required to pass the amount of water credited to these meters, one cannot fail to be impressed with the performance of these simple little machines, which seldom excite our admiration, and sometimes come in for severe condemnation when one's water bill happens to exceed his expectations.

In connection with other experimental work the author recently had occasion to test two 3-in. hot-water Worthington meters for the Cincinnati Water Department. These meters were used with water at 190° Fahr., of the natural consistency of the Ohio River, between July, 1897, and April, 1898. The back pressure on the meters was about 105 lbs. during service, and the rate of operation quite uniform from day to day, averaging 180 cu. ft. per hour. The results of these tests, at different periods in the service of the meters, indicate how rapidly they depreciated with hot Ohio River water. The pistons and cylinders were of brass, and the meters were constructed and reconstructed for the special service in which they were used.

TABLE No. 14.

3-In. Worthington Meter, No. 72 088.

3-In. Connections. Brass Pistons and Sleeves for Hot Water.

Meter reading.	Cubic feet for interval.	Ratio of tank to meter.	Error of meter. Per cent.
112 335	.....	1.0914	— 8.37
114 952	2 617	1.0958	— 8.74
139 484	24 532	1.1064	— 9.62
167 897	28 413	1.1146	— 10.28
198 987	31 090	1.1413	— 12.38
202 564	3 577	1.1831	— 15.48

3-In. Worthington Meter, No. 70 577.

3-in. Connections. Brass Pistons and Sleeves for Hot Water. (New.)

3 652	.....	1.0134	— 1.32
35 403	31 751	1.0309	— 3.00
61 256	25 853	1.0546	— 5.18
72 237	10 981	1.0729	— 6.80

3-In. Worthington Meter, No. 72 088. (Rebuilt.)

0 000	.....	1.0625	— 5.88
14 840	14 840	1.1226	— 10.92
18 209	3 369	1.1398	— 12.27

3-In. Worthington Meter, No. 72 088. (Again rebuilt.)

1 355	.....	0.999	+ 0.10
17 480	16 125	1.0133	— 1.31

The wear of meters with water at normal temperature, free from sand or other material which will cut the working parts, should be much less than with hot water containing at times much sand and mud in suspension, but the wear and reduced accuracy of water meters when used for a length of time, or after a reasonable service, is a matter to which little attention seems to have been given. Some unofficial reports have come to the author indicating that certain meters have passed enormous quantities of water without evidence of wear or reduced accuracy of measurement, but the water meter is a working machine, and, like all other machines, must be subject to wear, and, unlike most other machines, is not usually situated so as to admit of frequent examination and adjustment to compensate for wear. The author inclines to the opinion that the rate of wear and reduced accuracy of measurement will conform roughly to curves such as are indicated by Fig. 3 for the hot-water Worthington meters. The curve, of course, is to be fitted by experiment to each type, and perhaps each make of meter, and the water with which it is working.

In Fig. 3 the ordinates represent the percentage of delivery of the meter in excess of the registered amounts, or the percentage of water by tank measurement to the registry of the meter, less 100. In these experiments from 20 to 25 cu. ft. of water were drawn into the measuring tank, and the net weight of the contents of the tank was reduced to cubic feet for the observed temperature of the water as it passed through the meter.

If the author's theory is tenable, then meter manufacturers could furnish their customers with diagrams to be used from time to time in

DIAGRAM SHOWING RATE OF WEAR OF 3 IN. WORTHINGTON HOT-WATER METERS AT THE HUNT STREET PUMPING STATION, CINCINNATI, OHIO, WATER-WORKS. 1897-98.

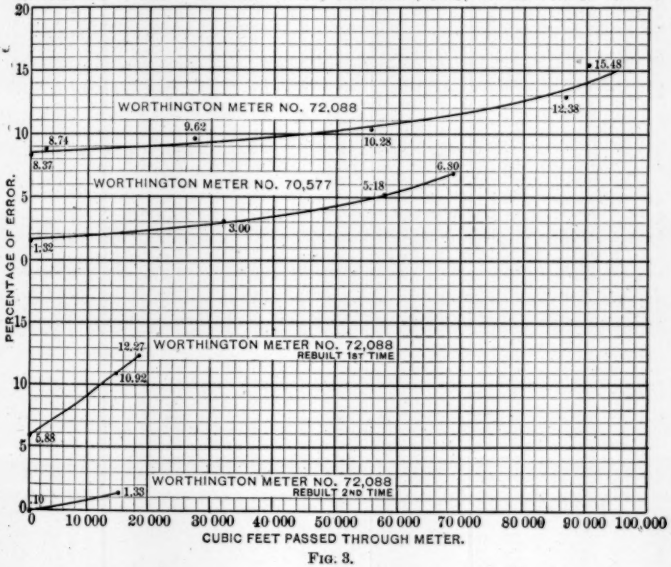


FIG. 3.

correcting the meter record, to obtain the true discharge or delivery of water. If a meter thus is fairly accurate when started, and in error —2% after 50 000 cu. ft., or some other quantity, has been passed through it, and out — 4 or — 5% after 100 000 cu. ft. has been passed through it, this diagram will furnish the proper factor by which to multiply the meter record at any period of its useful history.

This brings up the consideration of the maximum average error allowable in water meters. Conferences with several meter manufac-

turers indicate that in their opinion a meter is safe to use until the error reaches 7%, but there can be no doubt that there are in daily use many meters in which the average error is much greater than 7 per cent. Certain meters in these tests have average errors of more than 7%, and with meters which have been long in service, with no complaints as to stoppage or interference with the supply of water, the error, in many instances, as indicated by the author's experience, will be found to be much more than this modest figure. If 7% is regarded generally as the maximum allowable error, then thousands of meters now in use in all probability should be replaced by new meters, a proposition which all meter manufacturers doubtless will approve. An error of 10% may be admissible, provided it is known and the registry therefor can be corrected, while admitted errors greater than 10% unfortunately furnish material for condemnation of the meter system.

Construction Features of Water Meters.

In the rotary meters the pistons are made of vulcanized rubber. The Worthington pistons are made of drawn-brass tubing, with a central cast-brass head. This head unites the two tubes which form the piston, and receives the impact when the motion of the piston is arrested at the end of the stroke by the buffers. In the Crown, Empire and Union Rotary meters, the piston box or case is made of fine bronze. The piston box in the Hersey meter is made of vulcanized rubber, and the top and bottom plates which close the box are made of bronze and covered with a coating of rubber about $\frac{1}{16}$ in. thick. The working parts (excepting the gear-train) in this meter present only rubber surfaces to the action of the water. In all the disc meters, the discs are made of vulcanized rubber. In some meters the discs are flat, in others concave. If either form has an advantage over the other, the fact is not apparent to the author. The buffers of the Worthington meter are made of vulcanized rubber; in fact, this material forms such an important feature of water meters that in the Union Rotary and Columbia meters, even the gear-trains are made of vulcanized rubber; while in the Pittsburg meters, the first spindle of the gear-train turns in a vulcanized rubber bushing. The two large elliptical gears which connect the two pistons of the Union Rotary meter are made of vulcanized rubber, while the eccentric moving piston of the Empire meter is guided by a vulcanized rubber pin

which projects from its under side into a chamber in the lower cap of the piston box and impinges against a bronze friction roller.

With the exception of the Worthington, Union Rotary and Columbia, all the meters have bronze gear-trains, with cut or pressed wheels and pinions. The Worthington main gear is a single brass ratchet or crown wheel actuated by a pawl which is moved forward one tooth or more for each complete motion of the two pistons. In all the meters the internal screws are of brass. The disc case or box in all the disc meters examined is made of bronze. In the Trident meter the two sections of the disc box are united with a snap joint. In the Standard meter this snap joint is supplemented by screws, and in all others, excepting the Nash, the disc box is made up with screws. The bottom of the Nash disc box is held in place by friction, until the box is screwed down on the bottom part of the meter case. In the new form of Pittsburg meter the disc box forms a part of the meter case.

The bolts and nuts of the Trident, Hersey, Hersey Disc, Niagara and Pittsburg meters are of iron and galvanized; in all other meters, excepting the Worthington, the bolts are of iron with brass nuts. The Worthington meter case is put together with iron tap bolts.

Screens are provided in all the National meters to intercept solid particles in the water more than $\frac{1}{8}$ in. in diameter. The same result is accomplished in the Hersey Disc meter by the narrow annular space between the meter case and disc box through which the water passes before it reaches the disc box. In the Trident meter a strip of thin brass is attached at one end to the inside of the meter case, and extends partially around the disc box. This strip of brass is narrower than the chamber in the meter case and forms with the case a thin curved slot through which the water must pass before it enters the disc box. This device acts as a perfect interceptor of large solid particles which might enter the meter and interfere with its operation. No provision in the other meters to intercept floating solid matter was noticed by the author; but it is good practice with all meters to place a fish-trap or screen in the service pipe near the inlet connection, because such traps are more convenient to open and clean than is the meter itself.

Some complaint has been made of the discs of disc meters breaking under high pressures. If experience has demonstrated this to be an inherent defect of this type of meter, the difficulty might be

remedied by thickening the disc at the center and tapering it toward the edge. To do this, however, will require the sacrifice of a part of the ball-bearing surface or arc of vibration, which perhaps the meter manufacturers might consider a greater evil than the occasional breaking of the discs under high pressures. In the Nash meter the disc is strengthened by a steel wire, embedded in the rubber near the edge of the disc.

In these experiments the disc meters were frequently operated under pressures of 95 to 100 lbs. on the inlet side, and, to test the strength of the discs, this full head was often turned on the meter as suddenly as the cock in the supply pipe could be opened, with no evidence of injury to any meter which the author examined.

Frost bottoms are provided in the Trident and in the new form of the Pittsburg meters. These are of cast iron, inexpensive and easily replaced should an accident happen to the meter by the freezing of water in it. In other meters the top or bottom piece of the meter case is made of cast iron, which is expected by the manufacturers to break if the meter should freeze.

With one or two exceptions, all the meters examined were well built, with a nice fitting of all parts, and neat, legible dials. The meters generally throughout indicate the highest class of workmanship, and a careful consideration for the two essential features of a water meter, viz., accuracy of measurement and durability.

A function of the disc meter, which the author has not heretofore seen mentioned, is its capacity as a leak detector. All the disc meters which were placed in the line of the domestic service pipe, where the tests were conducted, maintained a continual vibration, which could be detected on the second floor of the building; while on closing a stop-cock in the discharge pipe near the outlet of the meter, the vibration ceased. An investigation of the cause of this continual and regular pulsation of the meter led to the discovery of trifling leaks in two taps which before had passed unnoticed. Upon timing the Nash meter under this condition, it was found that the vibrations of the disc were exactly four per second. The leaks were too small to be registered by the meter, and the only evidence it gave of their existence was the gentle vibration of the disc on its ball-bearing and the tapping of the driving pin against the guide or driving arm of the gear-train. After the discovery of this new feature in the disc meter,

several were connected one by one in the service pipe with similar results. If this rocking of the disc on its bearing with streams so small as to be unmeasurable is a common property of all disc meters, then they have an advantage over all other forms of meter, viz., of imparting knowledge of leaks in the domestic plumbing which otherwise might escape detection.

Revolutions of Pistons or Discs per Cubic Foot of Water.

After the several tests were made, all the meters were taken apart and carefully measured for water displacement per revolution of piston or disc, and the revolutions of piston or disc, and by gear-train, computed per cubic foot of water registered. For the piston meters this was determined by direct measurement of the working parts, while for the disc meters the displacement per revolution was obtained by carefully waxing all openings and filling with water the disc box with the disc in it. From the weight and temperature of the water, the net volume of the disc box was calculated. In several instances this result was checked from careful measurement of the disc and box. In the Columbia meter the revolutions per cubic foot of water discharged were calculated from the gear-train alone.

TABLE No. 15.—Revolutions of Pistons per Cubic Foot.

Meter.	Cubic inches, displacement per revolution.	Revolutions, by piston displacement.	Revolutions, by gear-train.	Percentage of slip allowed.
Piston Meters.				
¾-in. Worthington	56.544	30.56	30.00	1.83
¾-in. Crown	10.600	163.03	163.89	None.
¾-in. Hersey	15.750	109.71	109.37	None.
¾-in. Empire	6.359	271.72	268.94	1.03
¾-in. Union Rotary	17.360	99.54	98.65	0.90
Disc Meters.				
¾-in. Nash	3.869	446.63	446.75	None.
¾-in. Hersey	5.415	319.12	316.18	0.92
¾-in. Trident	4.770	362.32	358.40	1.08
¾-in. Pittsburg, 8 011	5.740	301.06	296.78	1.76
¾-in. Pittsburg, 10 798	5.876	294.10	289.06	1.72
¾-in. Niagara	6.285	274.95	268.75	2.26
¾-in. Standard	4.764	362.70	348.21	4.00
¾-in. Lambert	5.633	306.75	302.40	1.42
Inferential Meter.				
¾-in. Columbia	.....	.....	800.00	....

Some objection may be raised to the general crudeness of the apparatus used in the author's tests, but originally only two questions

were presented in connection with the introduction of water meters by his village: (1) The relative accuracy under the same conditions of pressure and water delivery of the cheaper disc meters when compared with the piston meters, and (2) the relative value of both types of meters when operated with widely varying streams of water. These questions the author believes have been answered by the tests.

The conditions were alike for all meters, excepting the pressures in the supply pipe, and this variation of pressure was due to the circumstance that the street main from which the supply pipe to the meters was run was also a pumping main, and the variation of pressure in the supply pipe could only have been avoided by making all the tests at night under reservoir pressure. The author has no reason to believe that this variation of pressure on the inlet sides of the meters had any influence on their accuracy of measurement. Results were sought which would be fairly comparable with each other rather than precise in themselves; but this statement must be accepted as applying to the apparatus and not to the observations and deductions, which were carefully made and checked by at least two competent persons.

Prior to this investigation of water meters for domestic service, the author has several times been called upon in a professional capacity to examine and report upon some of the better known forms of meters, to settle disputes between water-works officials and dissatisfied water takers. From this experience, and the occasional use of water meters for test purposes, he has acquired a confidence in the reliability of these devices, which, unfortunately, is not generally shared by water-works engineers. This confidence has been strengthened by the present investigation to such an extent that he believes the use of water meters should become, at the earliest possible date, an essential feature of all water-works which pretend to conduct business upon modern enlightened principles.

DISCUSSION.

JOHN THOMSON, M. Am. Soc. C. E.—This paper, treating of a test of commercial apparatus, is of the kind that almost any member of the Society has material for at different periods of his experience, but the fact is, that few members take the trouble to prepare and submit such useful data. The author is therefore to be especially commended for its presentation. Mr. Thomson.

The speaker regards the apparatus described as adequate for such a test as the author desired to make, and believes that the illustration, Fig. 2, might well serve as a model for those who are not satisfied with the experience of others and desire to demonstrate for themselves, through the medium of a bench test. As a manufacturer, the speaker prefers the use of tanks which indicate the quantity by volume rather than by weight, as less time is required, variation due to temperature need not be regarded, there is less likelihood of errors of observation, and the sliding scale which is connected to the glass indicating tube may be graduated to denote directly the percentage of the total quantity discharged through the meter. Thus, if the meter indicates 9 cu. ft., and 10 cu. ft. are found in the tank, the scale would read 90%; or, if the meter indicated 10 cu. ft., and 11 cu. ft. were found in the tank, then the scale would read 110 per cent. This system of notation is believed to be preferable to expressing the error as plus or minus, or by the proportion of cubic feet or gallons.

The author specifies and carries through his tables a refinement which, in the speaker's judgment, in a test of this character, is "more nice than wise," namely, the calculation of the weight of the water due to its temperature. This is referred to, not to be hypercritical, but simply to suggest that future experimenters may safely neglect this detail, because no commensurate result is ordinarily derived. Thus, in Table No. 1, the greatest difference of temperature noted is 9°, equivalent to approximately 0.05 lb. per cubic foot, or less than $\frac{1}{10}$ of 1%, an error which is probably less than that of the platform scale used. There is, however, an element of variation due to changes of temperature, within ranges of, say, 15 to 25°, which may be taken into account, under certain conditions of operation, that is, the effect upon the displacing member of the meter when made of hard rubber, as the coefficient of expansion of hard rubber is different from that of brass or composition. Therefore, a relatively moderate change of temperature may affect considerably the registration of the meter at low rates of flow, increasing or decreasing, as the case may be, its sensibility, and in cases of extreme high temperature this may wreck the meter.

The author occasionally uses the term "accurate" where "sensibility" would seem to more clearly convey the intended meaning. A

Mr. Thomson. meter may have a more or less wide range of sensibility without affecting its general efficiency, or accuracy, as a commercial instrument. Then, too, the maintained sensibility of a meter is dependent largely upon the conditions of operation and the water which passes through it. In this connection it may be well to note that the foreign matter carried by the water described by the author is not calculated to improve the condition of an accurately constructed device. As a matter of fact, as frequently proved by practical use, all meters set where the water is charged with grit or silt ought to be fitted to operate with a much less degree of sensibility than where the water is pure. The reason for this is that when the parts are closely fitted the first effect of grit is to cause them to wedge, bind, cut and wear, usually producing by such automatic action a greater degree of clearance than if originally fitted with sufficient freedom to permit the particles to pass freely. The speaker, however, has quite given up the expectancy that this oft-repeated suggestion will receive much attention; as ordinarily the water-works superintendent or engineer, who undergoes a change of heart from the system of disposal at the discharging capacity of the faucet to that of paying by measurement, is then usually not content to gauge his *aqua pura* by units of cubic feet or gallons; he at once rises to the situation and would subdivide it by drops, as if it were liquefied oxygen or Scotch whiskey.

The matter of sensibility, however, is especially important, and should be considered, where meters are used in connection with tank service and the like, or where driblet flows are desired to be registered. In the disc type of meter, especially in the smaller sizes, the degree of sensibility obtainable is dependent upon the quality of the water, that is, whether pure or otherwise, the materials used in construction, the frictional resistance of the reducing gear-train and the degree of accuracy with which the disc and its casing are fitted. Suffice it to say that it is entirely feasible, and has frequently been carried out in practice, when required, to produce disc meters of the $\frac{5}{8}$-in. and $\frac{3}{4}$-in. sizes that will indicate from 50 to 75% of the quantity discharged when the rate of flow is as low as $\frac{1}{10}$ cu. ft. per hour; but, for the great majority of conditions, such refinement would be as much out of place as to nickel-plate a sand pump or a soldering iron.

The comparison of the Wyoming and Hamburg tests, Table No. 8, is interesting as showing the wide range in rate of flow, from 20 to 1 up to 119 to 1.

Regarding the matter of obstruction, or loss of head due to the meter, this, in many cases, is of but little practical importance. The general application of meters, with the resultant saving of wasteful flow, has the result of increasing the dynamic head. This is a well-known fact, but is not infrequently overlooked by experimenters and theorists. In so far as meters of the disc type are concerned, all of those now on the

market could readily be modified to secure a delivery almost equal to that of the pipe, but this would be bad for some of the meters. Nearly all the differences of obstruction in the several disc meters tested by the author are due, not to differences of mechanical operation, but to variations in the form and construction of the water-ways and ports, and the screens or other analogous means for preventing material of large caliber from entering the measuring chamber. Strange as it may seem to some, the fact that a meter may be designed so as to be very free in its delivery may, in some instances, be an element of danger, in that it will more quickly result in the destruction of the meter. The reason for this is that it is only in rare instances that meters are selected, for the service they are to perform, on the basis of their capacity. The almost universal practice is to simply select a meter, the nominal size of which corresponds to that of the pipe to which it is to be attached. In other words, pipe-fitters and political appointees are not always engineers, and a meter, to be successful commercially, must be designed to meet the conditions just intimated. Mr. Thomson.

In fine, the nub of this obstruction matter may be thus illustrated: First, assume a water supply in which the quantity and the head are just sufficient to furnish a certain number of services, and that these services are to be metered. Then, if the meters themselves caused an obstruction to the flow greater than that of the pipe which they displaced, the desired supply, obviously, could not be obtained. This would be a condition where there might be justification for eliminating loss of head to the last place of decimals. But, second, no such condition has to be met in regular practice, this kind of "practice" being where meters are set to restrict waste. In such cases, whether the coefficient of obstruction of the meter is a full number or a fraction thereof is of but little consequence, in that the quantity saved will restore a greater degree of the dynamic head than is absorbed in the measurement of the quantity used. This fact has been proven so completely and so frequently that an excuse would be made for bringing it up here were it not a theme which, dear to the hearts of many, like the problem of squaring the circle or soliloquizing on a new spacial dimension, will not down. The hardship of such considerations is not to the manufacturers, who may be assumed to know something of the necessary requirements, but to the investigator whose mind may thus be clouded and perplexed by attempting to assimilate data which are largely nonsensical and useless.

The method and means which the author used for ascertaining the obstruction produced by the meters—the introduction of a section of service pipe corresponding in length with that of the meters—the speaker regards as most excellent, he having used this for similar tests with the utmost satisfaction; but he fails to see why a comparison of the discharge of the meter should have been made with curves of theo-

Mr. Thomson.

retical discharge, when, if the speaker understands the data correctly, a comparison should have been made directly between the discharge of the meters and that of the pipe. Referring to Table No. 6, and taking the discharge in gallons per hour, through the $\frac{1}{2}$-in. orifice, the average of the first five quantities in the right-hand column is 961.49 galls. Then, taking five of the disc meters tested, as in Table No. 7, the maximum average is found to be 906.21 galls., or, assuming the pipe as unity, as it is for this purpose, and the static pressure equal in both cases, then the relative capacity of the meter is 94.2%; that is, when the maximum discharge is determined, both in the meters and in the pipe, by the same static head and the same sharp-edged diaphragm having a perforation $\frac{1}{2}$ in. in diameter.

Supplementing the remark about the test by the Boston Water Board, it may be of interest to note that no meters of the disc type were then on the market; and that since then, or in about eleven years, fully 250 000 meters of this type have been made and sold by American manufacturers. Had it not been for archaic conservatism this number would easily have been doubled. The future, however, bids fair to correct this in full measure. The speaker, therefore, takes the liberty of asserting that the question of type is not a matter yet to be demonstrated; as the disc type, to quote, by adoption, the author's own words, has already "become the standard of water meters for domestic uses." Therefore, in this regard, the only feature of consequence, which for several years has received the close consideration of engineers familiar with the state of the art, has been the differentiation of the type to its lowest terms as respects price, accuracy, general efficiency and cost of maintenance.

Regarding the relative importance of durability: This is by far the most important element to be considered. It is taken for granted that the author includes in this general term convenience of examination, maintained accuracy, immunity from stoppages and cost of maintenance. A rather conclusive proof of the durability and continuity of performance of meters of the disc type is furnished by the author himself, in that the four Trident meters specified by their serial numbers in Table No. 1 were shipped to Wyoming in September, 1896, where they had, it is understood, been placed in service, and that at least one of them, No. 23 176, had passed, as the speaker is informed by the author, nearly 25 000 cu. ft. of water prior to removing it for this bench test. Moreover, the original calibration, as recorded at the factory, corresponds closely, at maximum flows, with those given in the table. The sensibility, however, shows considerable deterioration. Nevertheless, if the author can say that all these meters had been continuously in service, then their performance on sensibility flows may be regarded as highly creditable, especially so, if they were not pulled apart and cleaned out prior to the test, so as to put them in proper condition for a comparative

demonstration with new meters. Moreover, these meters, as originally furnished, were provided with extension dials about 3 ft. in height. It would also be interesting to know, in this connection, whether these meters were tested with or without the extensions. If without the extensions, and the meters were subsequently set in service with the extensions, then the bench test for sensibility would have but little value, as the extra friction caused by the additional parts materially reduces the sensibility of all meters arranged in this manner. Mr. Thomson.

The experiment of turning on the full inlet pressure suddenly "to test the strength of the disc," is without significance; having, as a demonstration, no practical value whatever. This statement is made on

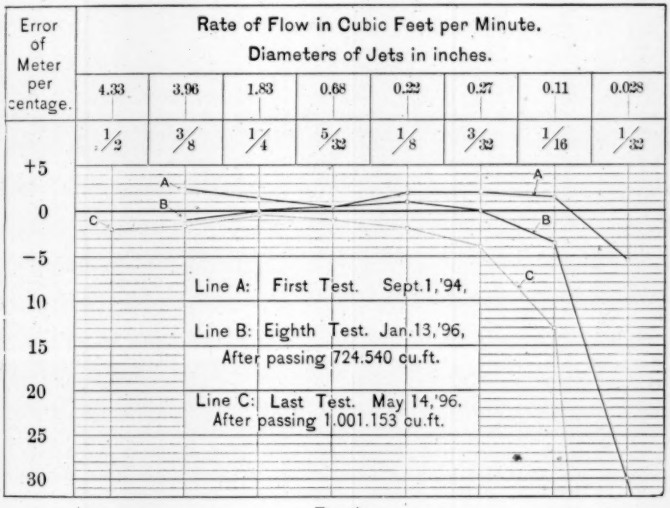


FIG. 4.

the presumption that the meters were tested in the circuit as shown in Fig. 2. The intention of this experiment, however, was most laudable; for one of the most serious defects of the disc system is the liability of the disc-piston to break, especially when made of hard rubber. Such failures, however, are quite as likely to occur when the pressure is 40 or 50 lbs. to the square inch as when it is 150 or 200 lbs. Fractures of disc-pistons are not due merely to excessive pressure, but are caused by such a hydraulic condition as will produce very high velocity of flow through the disc-chamber, thus operating the disc at a much higher rate of speed than that for which it is properly adapted, and hence in-

Mr. Thomson.

ducing rapid wear of the ball and socket, the periphery of which, in nearly all the meters cited, will permit the disc to jam between the wall of its casing and the diaphragm, which results in its ruin. It would appear to be evident, therefore, that a mere bench test is of but little value for ascertaining the relative durability of disc meters, in respect to the endurance of the discs, and that for this detail the only test entitled to any importance is to operate the meters under very high rates of delivery, either continuously or intermittently, until large volumes of water have been discharged. The consequence of such a trial will be either to smash the disc or show by its appearance its ability to stand severe punishment. As bearing upon this question, the results of some experiments of the nature indicated, made by Mr. A. W. F. Brown, Water Registrar of Fitchburg, Mass., are shown in Table No. 16, and in Fig. 4, which the speaker has prepared from original data furnished by Mr. Brown.

TABLE No. 16.—Durability Test of a $\frac{5}{8}$-In. Disc Water Meter, prepared from Records furnished by Mr. A. W. F. Brown, Water Registrar, Fitchburg, Mass., who Conducted and Controlled the Performance.

(This trial was undertaken with the intention and expectancy of destroying the meters.)

Dates.	Quantity passed, in cubic feet.	Rate of flow, in cubic feet per minute.	Diameter of Jets, in Inches, and Percentage of Error, Plus or Minus.						
			$\frac{3}{8}$	$\frac{1}{4}$	$\frac{5}{32}$	$\frac{1}{8}$	$\frac{3}{32}$	$\frac{1}{16}$	$\frac{1}{32}$
Sept. 1, 1894..	.....	.....	+2.5	+2.0	+0.5	+2.0	+2.0	+ 1.5	— 5.5
Sept. 19, 1894..	100 772	4.0	+1.0	+0.1	+0.5	+2.0	+1.0	— 2.0	—32.0
Oct. 3, 1894..	205 145	6.0	0.0	+0.5	+1.0	+1.0	—2.0	— 7.0	—33.0
Jan. 28, 1895..	327 888	1.5	+1.0	+0.5	+1.0	+1.0	+2.0	— 4.0	—30.0
April 18, 1895..	409 037	1.5	+2.0	+2.0	+2.0	+1.0	0.0	— 3.5	—30.0
Oct. 23, 1895..	631 761	1.5	0.0	—0.8	+1.5	—0.5	—1.0	— 5.0	—33.0
Jan. 13, 1896..	724 570	1.5	—1.0	0.0	+0.5	+1.0	0.0	— 3.5	—30.0
April 25, 1896..	906 229	1.5	—0.5	—0.5	0.0	—0.5	—3.0	— 9.5	.....
May 14, 1896..	1 001 153	4.5	—1.5	—0.5	+1.0	—2.0	—4.0	—13.0	.....
Maximum errors.....			+2.5	+2.0	+2.0	+2.0	—5.0	—13.0	—100.0
Greatest variations.....			4.0	2.8	3.0	4.0	6.0	14.5	94.5
Mean errors.....			1.0	0.8	0.9	1.2	1.6	5.4	27.5

Mean error of all service flows, from $\frac{1}{16}$-in. up..... 1.1%

Quantity in United States gallons..... 7 508 647
Revolutions of disc..... 358 873 235
Duration of trial, about 1 year and 8 months.
Equivalent mechanical service, assuming daily consumption of 250 galls..... 113 years.

Concurrent with the above, another $\frac{5}{8}$-in. Disc Meter was also tested until 817 561 cu. ft. had been delivered; the last 78 000 ft. being at the rate of 6.5 cu. ft. a minute under 150 lbs. pressure, finally smashing the disc, which, after passing about 740 000 cu. ft., had stood for 27 300 000 revolutions, at the rate of 38 nutations a second.

Mr. Thomson.

Regarding the author's theory as to rate of wear, as illustrated in the diagram, Fig. 3, in the speaker's judgment this is entirely untenable, and is not even properly applicable in the instance of the Worthington meter, to which the diagram refers, because a 3-in. meter of this type for the duty mentioned, is only worked to about $\frac{1}{5}$ of its maximum capacity. Therefore, if fitted to give sufficiently accurate results at so low a flow, under the conditions mentioned, it will the sooner wear itself free. This example is not properly comparable with regular practice, and does not do justice to the enduring quality of this meter.

The author's clever use of the disc meter as a leak detector is a novelty. The speaker once used a very sensitive meter for the purpose mentioned, by removing the gear-train and substituting a delicately adjusted electric contact device, the circuit being closed and opened by the nutation of the disc. The wires were conducted to the desired place of observation and connected to a telegraphic sounder, the time interval between the "clicks" indicating the rate of flow and the quantity. The number of places of decimals to which it would indicate was never definitely ascertained, but its sensibility was so pronounced that it was promptly christened the "dew-drop" meter.

The observation made as to the desirability of information respecting the accuracy of water meters after a period of use is fully concurred in by the speaker, who believes that much valuable information could be contributed by various members of the Society if they would do so. This information is largely in the hands of purchasers, and can only be furnished by them. Private corporations, for various business reasons, rarely publish such records; while the records of municipal departments are usually not in form to be readily available. This condition is improving, however, and many engineers of public works and water supplies are now keeping close tab on this feature and also on the cost of maintenance.

In conclusion, the speaker ventures to remark that the fact that water meters in general perform as well as they do is to him a matter of surprise; for there is no machine, or instrument, bar none, from which so much is preliminarily exacted, and which receives so little attention once it is set in the dark corner of a cellar or vault. When the policy of legitimate use shall have been substituted for that of unlimited waste, then this compact, inexpensive, ever-vigilant monitor, when properly utilized, is capable of saving to taxpayers during every decade tens of millions of dollars' worth of needless construction.

Mr. Smith.

J. Waldo Smith, M. Am. Soc. C. E.—At this time, when cities so quickly grow beyond the capacity of a water supply, which, at the time it was acquired, was expected to last for half a century, and when economy in the use of water is such an important factor, any information which adds to the knowledge of water meters is a blessing to the water-works engineer.

Mr. Smith. Table No. 2 seems to the speaker to be misleading and liable to give an entirely wrong impression unless studied with great care. As he understands it, a straight arithmetical average of the errors has been made, as shown in Table No. 1, without regard to the range of flow between the different experiments. In other words, an insignificant part of the range of the meter has been given equal weight with the conditions of flow which cover nearly the whole of that range.

Had lower rates of flow been tried, say $\frac{1}{64}$-in. orifice, the maximum error might easily have been 60%, and the average error 20%, and still, as ordinarily used, all the meters would have been practically correct.

Take, for example, the curve of velocities in a float measurement. It would be manifestly erroneous to average a velocity taken close to the side of the flume, and representing a width of 0.5 ft., with those taken some distance out, and representing widths of several feet, unless each were multiplied by the width it covered.

By this way of figuring, a meter which would not register with the $\frac{1}{32}$-in. or $\frac{1}{16}$-in. orifice would show a very small average error, and one which registered only about 40% of flow with a $\frac{1}{32}$-in. orifice, and which was tested with but two additional flows, would show a very large average error, and would give the impression that the meter is of no value as a measure for water, while for all ordinary flows it is almost exactly correct. This is one of the places where an average is wrong and is entirely misleading.

For similar reasons the comparisons given in Tables Nos. 3 and 4 are not fair, for in the Wyoming tests of $\frac{5}{8}$-in. meters the minimum flow was at the rate of about 11 galls. per hour, while in the Boston tests of $\frac{3}{4}$-in. meters, which are of about double the capacity of $\frac{5}{8}$-in. meters, the minimum rate of flow was about $8\frac{1}{2}$ galls. per hour, which is evidently unfair to the Boston tests if compared by average error, as in Table No. 4, and the last column in this table does not mean anything.

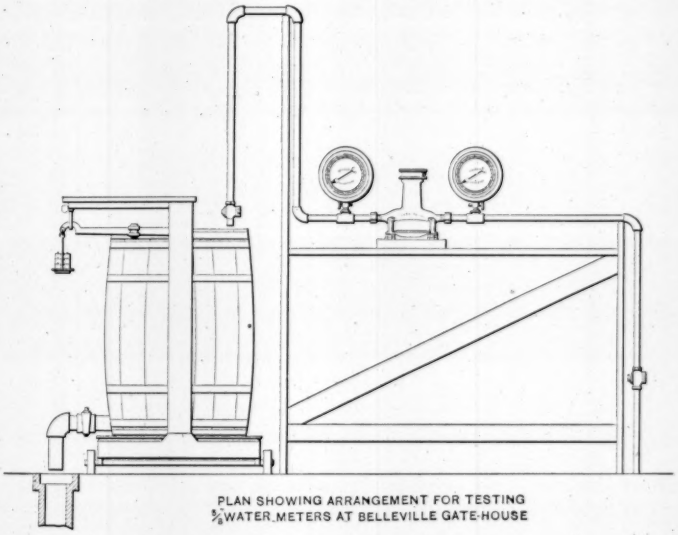
The last column of Table No. 5 gives the percentage of loss of head figured on the inlet pressure. Now, the loss of head is only indirectly a function of the inlet pressure, being rather a function of the rate of flow, and it is impracticable to compare tests of meters on this basis. The speaker regrets that this table is not in the form of rate of flow and loss of head.

The importance of the loss of head cannot be overestimated. Upon one hand the consumer usually objects to any appreciable lowering of the efficiency of his service pipe as a water carrier, and on the other hand the Water Department desires to set as small a meter as possible in order to reduce the expense. With meters showing a small loss of head, it is often possible to set meters of a smaller size than the service pipe, which would not be wise with meters showing a large loss of head. In reference to this, the speaker believes it is economy to put

the meter department in the hands of a competent engineer who is posted on the flow of water, and who will not always insist on setting a meter of the same size as the pipe when one much smaller is ample for the work, or *vice versa*. In the speaker's experience during the past year, a number of places have been found having 4-in. services where a 1-in. or $1\frac{1}{2}$-in. meter was ample to measure all the water used. Mr. Smith.

The speaker makes the same objection to Table No. 9 as to Table No. 4, and cannot see how a comparison can be made on the basis of average error, unless the tests are made at exactly the same rate of flow and the meters are of the same size.

In Table No. 10 the Wyoming, Hamburg and Boston tests are com-



PLAN SHOWING ARRANGEMENT FOR TESTING
⅝ WATER METERS AT BELLEVILLE GATE-HOUSE

FIG. 5.

pared by percentage loss of head. If the inlet pressure was the same in each case this might be possible, but with the inlet pressure varying in the different cases it is not practicable.

The speaker has never been able to make a satisfactory comparison in such cases as this without the aid of curves and diagrams, and regrets very much that these experiments have not been reduced to this form.

In regard to the curves of deterioration, as illustrated by the diagram of the Worthington meter, Fig. 3, the speaker is of the opinion that meters of the same make and using the same water would show deterioration curves widely different, and that such curves would not

Mr. Smith. be of any practical use unless the meter was run at a uniform rate, for the error of the meter depends on the rate of flow.

Aside from the foregoing the speaker regards the rest of the data and opinions presented as admirable. What is said in regard to the meter as a leak detector is entirely new, and should prove very useful. The speaker can say "Amen" most heartily to the author's closing remarks in regard to the general use of water meters, and he looks for the day when water will be sold entirely by meter measurement.

To compare the flow as measured by the meters with the total quantity pumped would be exceedingly difficult, on account of the many variable factors in the case. The slip of the pump and the leakage of the pipe system are usually unknown and variable quantities, and hence there is great uncertainty as to how much of the full quantity of water the meters fail to register. Possibly under a new order of things, when pipe systems can be divided into a number of parts, with each particular part of the system measured by one large Venturi meter, something in the line of Mr. Harlow's suggestion may be done; but at present the speaker does not see how it could be carried out. In a case in Burlington, Vt., some time ago, and which was reported to the New England Water-Works Association, a small district was entirely metered, and the leakage of the pipe system had been measured. On comparison it was found that the meters accounted for 90 or 91% of the total quantity supplied.

In view of the character of this paper and its probable value to water-works superintendents and engineers the speaker takes this opportunity to present in condensed form a set of experiments on small water meters made some years ago by the direction of Clemens Herschel, M. Am. Soc. C. E., Engineer and Superintendent of the East Jersey Water Company, in order to determine, if possible, the best meter for ordinary house service.

Seven meters were purchased in the open market without the manufacturers being aware of what was to be done with them. Four, the Union Rotary, Hersey Rotary, Crown and an experimental meter were of the rotary piston type, and three, the Thomson Bee, Hersey Disc and Trident, were disc meters.

The tests were all made in the Belleville gate house of the East Jersey Water Company. All the meters were first tested for accuracy. For test, a meter was set firmly in a horizontal position in a short line of ½-in. pipe which joined a 1-in. pipe connecting to one of the main 16-in. supply pipes, about 2 ft. up stream from the meter. On the down-stream side, the short piece of vertical pipe just over the measuring barrel was also 1 in. in diameter, and was terminated by a 1-in. stop cock. The rate of flow was varied by screwing plugs, bored with different-sized holes, into the stop cock, so that in all cases the 1-in. cock was open full during an experiment, the flow being throttled by

the plug. To regulate very small flows a ¼-in. pipe with a stop cock was attached to the 1-in. cock. Mr. Smith.

The measuring barrel was set on a Fairbanks standard platform scale and was perfectly tight. All weights were read to the nearest $\frac{1}{10}$ lb. The general arrangement is shown in Fig. 5.

On each side of the meter, and as near to it as practicable, a pressure gauge was attached to the ½-in. pipe, so as to observe the loss of head caused by the passage of water through the meter and its couplings. These gauges were frequently compared with each other, when under the same pressure, and were also rated on a Crosby gauge tester. Pressure was observed to the nearest $\frac{1}{10}$ lb.

The method of procedure in making a test was as follows:

Scale, with measuring barrel on, balanced, and weight recorded; reading of meter recorded; flow started, and time noted; reading of pressure gauges noted; after a sufficient quantity of water had passed for a good experiment, flow was stopped, time noted and water was weighed.

Except for the very small rates of flow, nothing less than 1 cu. ft. was passed through the meter for an experiment, and for the larger flows the quantity was from 2 to 5 cu. ft.

In the first set of accuracy tests, Fig. 6, about thirty experiments were made with each meter, and in the subsequent tests about twenty experiments.

The maximum pressure on the meters was from 65 to 68 lbs.

After being tested for accuracy, the meters were set up and run until their dials registered from 103 000 to 106 000 cu. ft., when they were again tested for accuracy.

The time required to register these quantities was from twenty-eight to thirty-five days. The rates of flow are given in Table No. 17.

TABLE No. 17.

Union Rotary..	0.043	cu. ft.	per second	= 19.5	galls.	per minute
Experimental..	.040	"	"	= 18.0	"	"
Trident.,	.039	"	"	= 18.0	"	"
Thomson	.039	"	"	= 18.0	"	"
Hersey Disc...	.039	"	"	= 18.0	"	"
Crown	.036	"	"	= 16.0	"	"
Hersey Rotary.	.034	"	"	= 15.0	"	"

The experimental meter failed after running twenty days and registering 70 000 cu. ft. The stoppage was due to the breaking of the rollers.

The meters were now tested for accuracy (Fig. 7), as in the first place, and then returned to their places, and run against time.

Mr. Smith. Where meters are in general use the consumption is said to be about 3 000 cu. ft. per family per year, which seems small. From the speaker's observations he would consider it safer to put the consumption at 10 000 cu. ft. per year in estimating the service of a $\frac{5}{8}$-in. meter.

In calculating the yearly service of meters, the speaker will assume that 100 000 cu. ft. is equal to 10 years' service.

Up to the end of the second durability test the meters had run as shown in Table No. 18.

TABLE No. 18.

Name of meter.	Total time run, in days.	Total quantity registered. Cubic feet.	Equivalent years' service on basis of 100 000 cu. ft. = 10 years' service.	Pressure maintained on up-stream side. Pounds.	Rate of flow. Cubic feet per second.	Remarks
Union Rotary....	73	266 000	26.6	51	0.042	
Experimental...	.....	.....	.....	.....	.....	Rollers broken.
Trident.....	77	267 000	26.7	46	0.040	
Thomson.....	77	264 000	26.4	50	.040	
Hersey Disc.....	77	264 000	26.4	48	.040	
Crown.....	76	238 000	23.8	56	.036	
Hersey Rotary..	80	235 000	23.5	51	.034	

The meters were now tested the third time for accuracy (Fig. 8), and were then run against time as before, until they had registered the quantities given in Table No. 19, when they were again tested for accuracy, the results of which are shown by the curves in Fig. 9.

TABLE No. 19.

Name of meter.	Total time run, in days.	Total quantity registered. Cubic feet.	Equivalent years' service on basis of 100 000 cu. ft. = 10 years' service.	Pressure maintained on up-stream side, in pounds.	Rate of flow. Cubic feet per second.	Remarks.
Union Rotary...	103	377 000	37.7	.....	0.042	
Experimental....	25	82 000	8.2	.....	.038	Broken.
Trident.....	108	377 000	37.7	.....	.040	
Thomson.....	108	376 000	37.6	.....	.040	
Hersey Disc.....	108	371 000	37.1	.....	.040	
Crown.....	106	325 000	32.5	.....	.035	
Hersey Rotary..	112	327 000	32.7	.....	.034	

They were now set in place and left standing under 65 lbs. pressure for four months, May 21st to Sept. 24th, 1895, and were then tried for

sensitiveness by turning the stop-cock in the discharge pipe of each until the meter just started, when the rate of flow was measured. The results are given in Table No. 20. Mr. Smith.

TABLE No. 20.—RATES OF FLOW AT WHICH METERS WOULD JUST BEGIN TO REGISTER, AFTER STANDING 4 MONTHS UNDER PRESSURE WITH NO WATER RUNNING THROUGH.

Name of meter.	Rate of flow. Cubic feet per second.	Rate of flow. Gallons per minute.
Union Rotary	0.00051	0.23
Crown	.00068	0.31
Hersey Rotary	.00126	0.57
Trident	.00047	0.21
Thomson	.00047	0.21
Hersey Disc	Played out, will not run.	

This test was not entirely satisfactory, although it is of some value in indicating to what extent the sensitiveness is affected by the meter standing still for a long time.

The five remaining meters were now set running as before, and at about the same rate.

Union: Continued to run until it was shut off, when it had registered 1 589 000 cu. ft.

Crown: Ran until shut off, and had registered 1 120 000 cu. ft.

Hersey Rotary: Stopped after registering 920 000 cu. ft., and was then completely worn out.

Trident: Continued to run without accident until shut off, when it had registered 1 644 000 cu. ft.

Thomson: After registering 711 100 cu. ft., stuck fast, but was repaired and started again, but stopped after registering a total of 932 000 cu. ft.

The total registration of the meters, the rate of flow, etc., up to the time each meter stopped or was shut off, is given in Table No. 21.

There is little doubt but that some or all of these meters would have stopped sooner if they had been run at a lower rate of flow, under a less head, or with roily water.

Up to the end of the test, no meter except the Thomson had been taken apart or had had any repairs whatever. During the entire period, however, the water was very clear, and contained little or no sediment.

After the test, as recorded, the meters were thrown to one side, and forgotten for about eighteen months. Two, the Trident and Crown, have recently been tested. At the time of this last test neither had received any particular care, nor had they been taken apart, repaired or renewed in any part.

The results of this last test are given in Table No. 22.

Mr. Smith.

TABLE No. 21.

Name of meter.	Total days run.	Total quantity registered. Cubic feet.	Equivalent years' service on basis of 100 000 cu. ft. = 10 years' supply.	Rate of Flow.		Remarks.
				Cubic feet per second.	Gallons per minute.	
Union Rotary.	413	1 589 000	158.9	0.045	20.2	Shut off, but is still in running order.
Trident.....	427	1 644|000	164.4	.045	20.2	Shut off, but is still in running order.
Thomson.....	268	932 000	93.2	.040	18.0	Stopped. Also stopped after registering 711 000 cu. ft.
Crown	406	1 120 600	112.0	.032	14.4	Shut off, but is still in running order.
Hersey Rotary	315	920 000	92.0	.035	15.7	Stopped. Worn out.

TABLE No. 22.

	Trident.		Crown.	
	Rate of flow, in cubic feet per second.	Percentage of flow registered.	Rate of flow, in cubic feet per second.	Percentage of flow registered.
Orifice full.....	0.0604	98.8	0.0357	67.5
$\frac{1}{2}$-in	0.0455	98.8	0.0269	68.0
$\frac{3}{8}$-in.....	0.0376	98.5	0.0175	68.8
$\frac{1}{4}$-in.....	0.0185	98.6	0.0049	Will not run unless shaken.
$\frac{1}{8}$-in.....	0.0051	96.5		
$\frac{1}{16}$-in.....	0.0016	40.0		

Will just run with $\frac{1}{16}$-in. orifice, and will not move with $\frac{1}{32}$-in. orifice.

The record of each of these meters is phenomenal. The Trident, between flows of 2.3 and 27.0 galls. per minute, gives an average accuracy of 98.2%, and is sensitive to a flow of 0.7 gall. per minute. The Crown does not give quite such good results, but in justice to this meter and other similar rotary meters, it should be stated that this form of meter can be restored to nearly its original accuracy and sensitiveness by turning down the case. In the present test it was desired to see the result of running the meters to destruction, without doing anything to them.

On examination, both of these meters seemed to be in fair condition, and with a few repairs would have stood a long run before failing. It would be interesting to repair these meters and see how much water they would pass before failing, also how nearly they could be restored to something like their original sensitiveness and accuracy at small expense.

Mr. Smith.

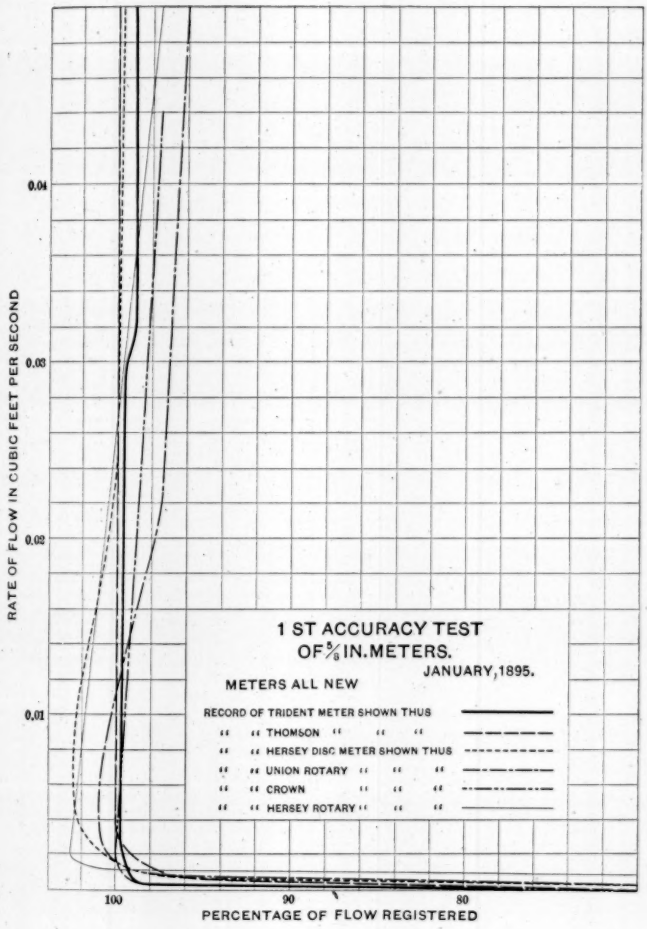


FIG. 6.

Mr. Smith.

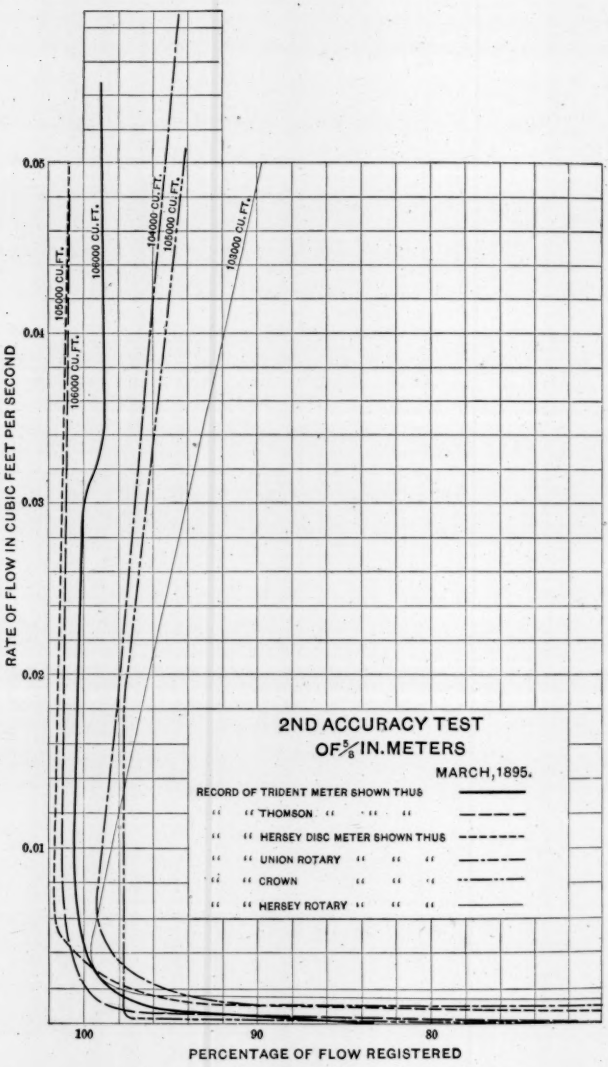


FIG. 7.

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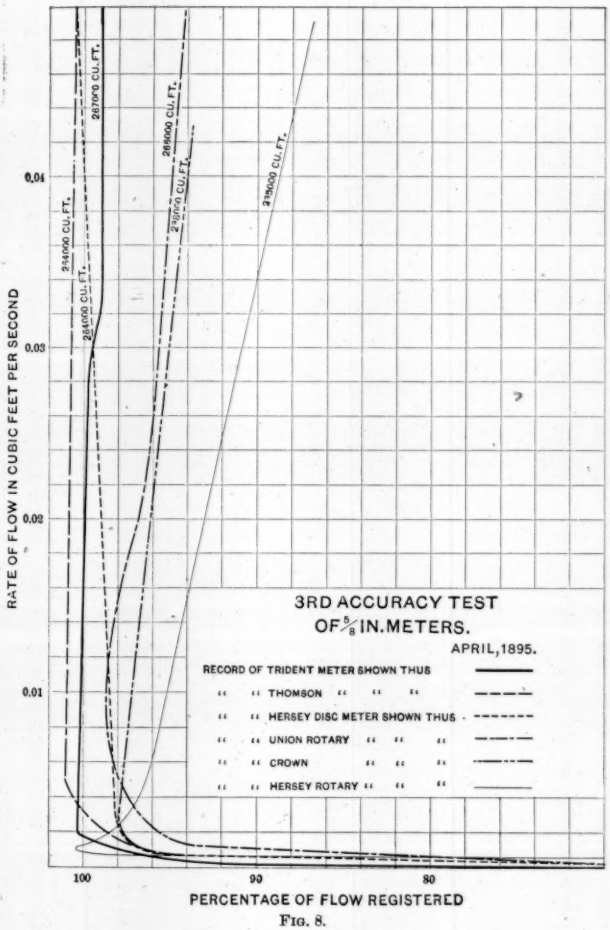

FIG. 8.

Mr. Smith.

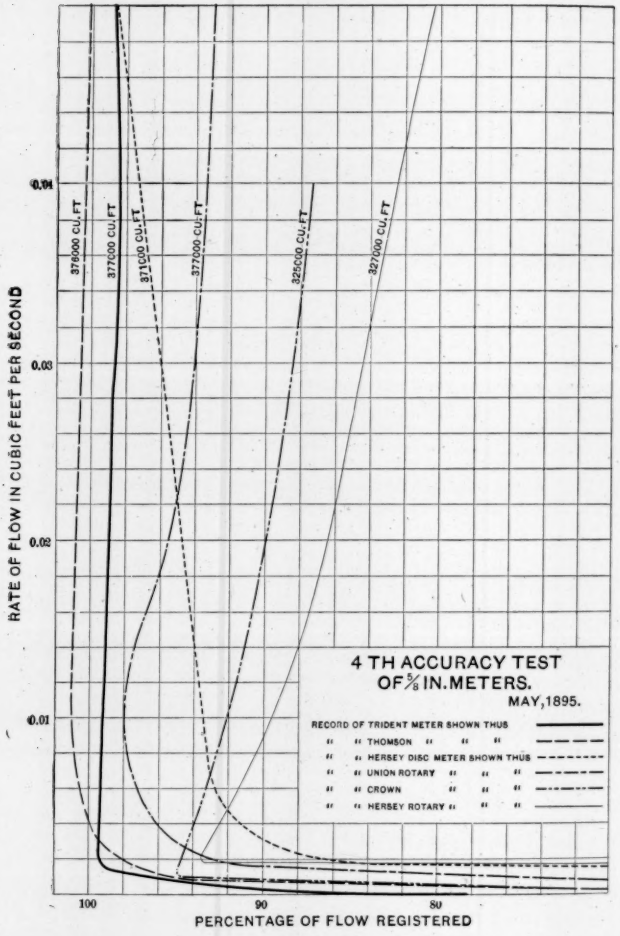

FIG. 9.

Mr. Smith.

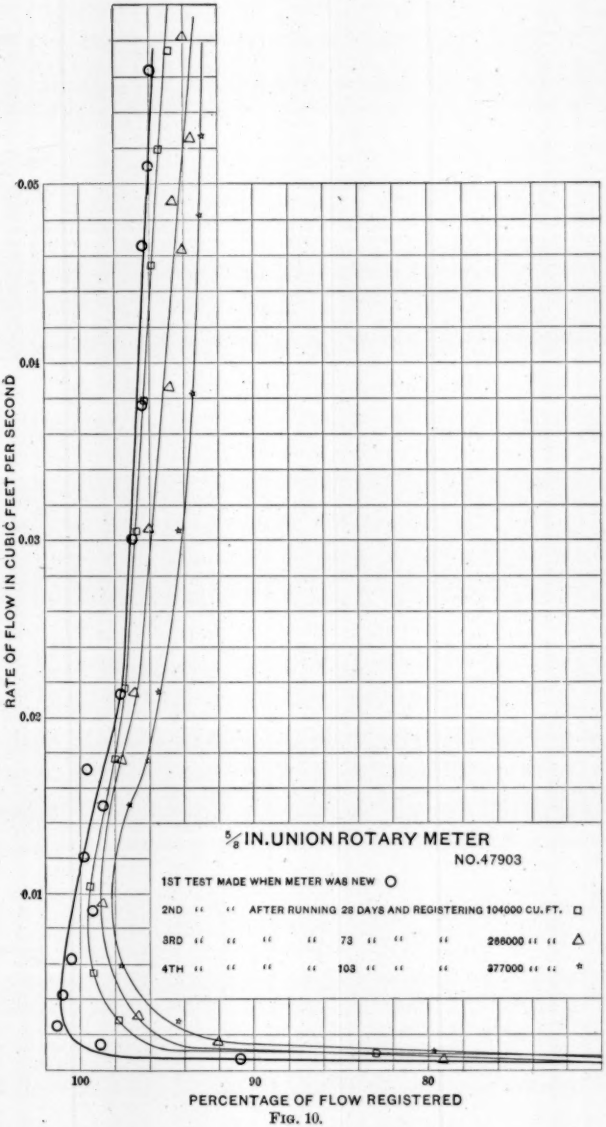

FIG. 10.

Mr. Smith.

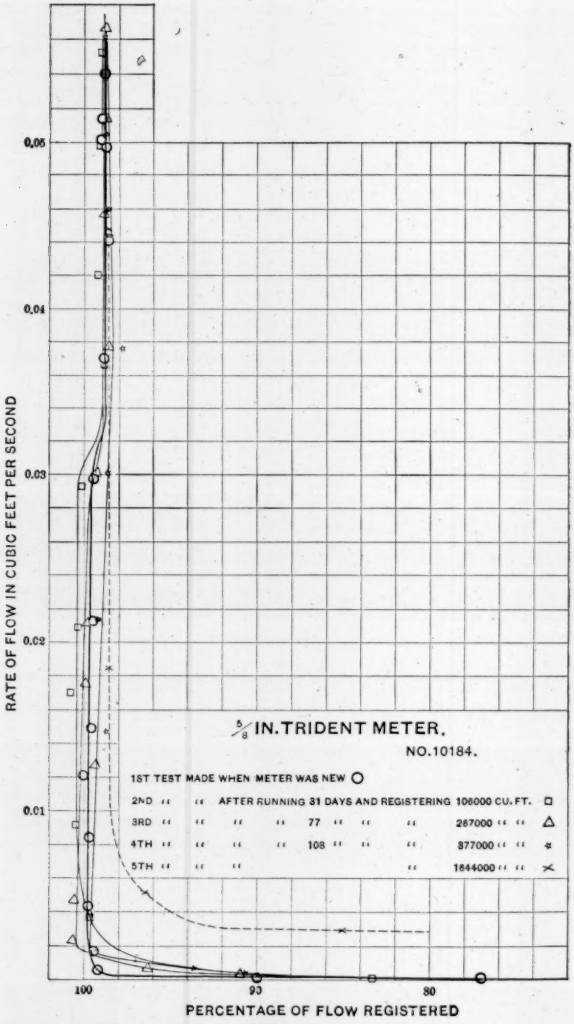

FIG. 11.

Mr. Smith.

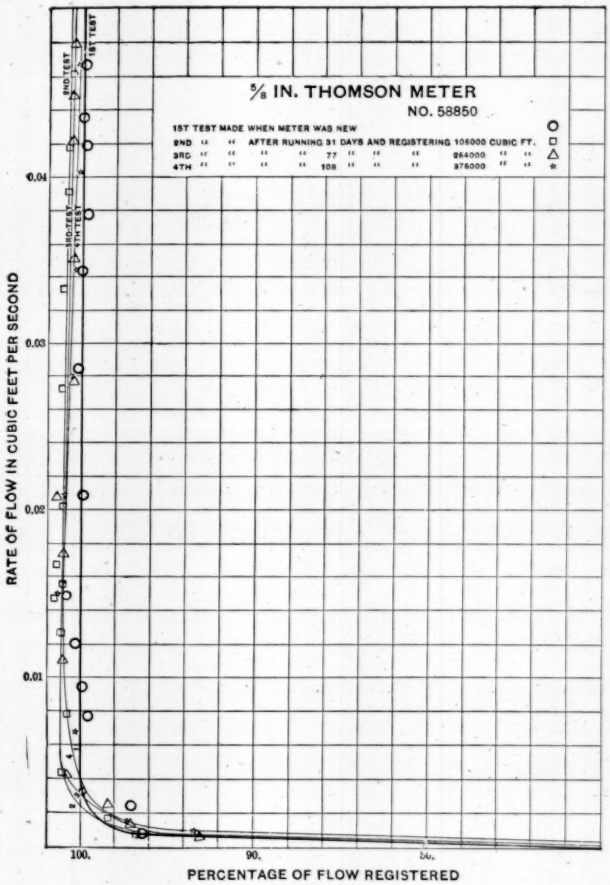

FIG. 12.

Mr. Smith.

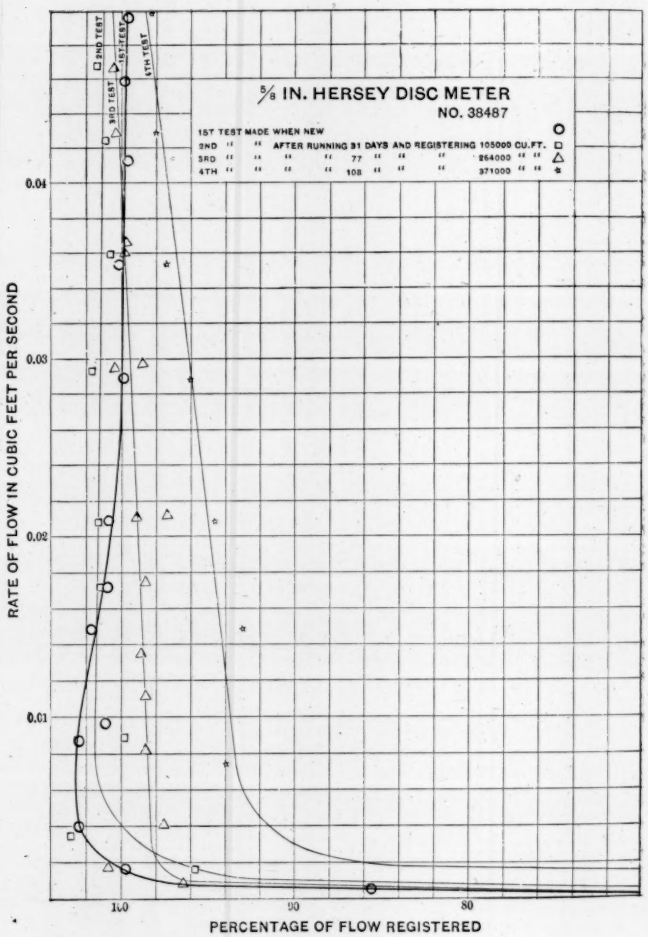

FIG. 13.

Mr. Smith.

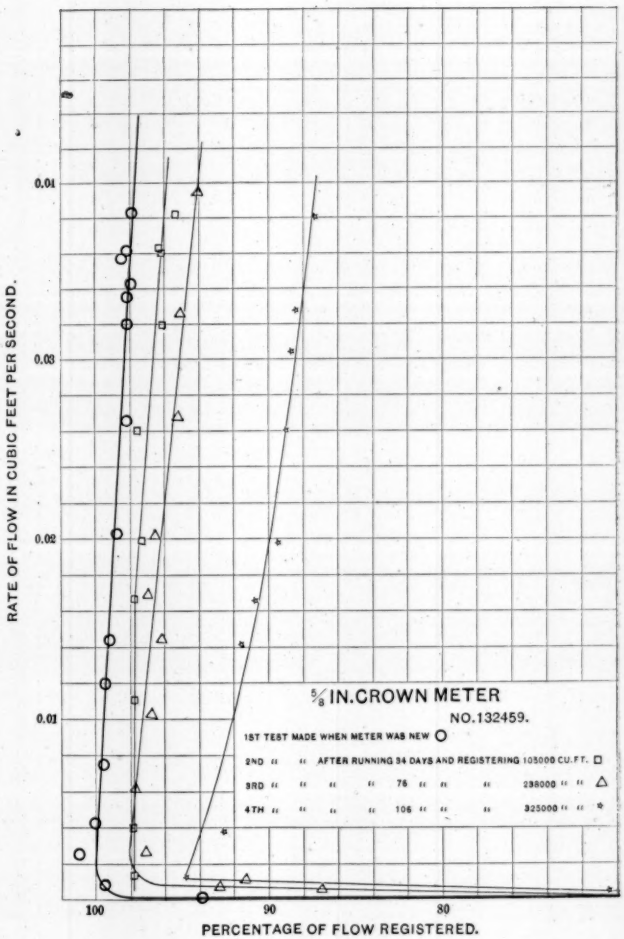

FIG. 14.

Mr. Smith.

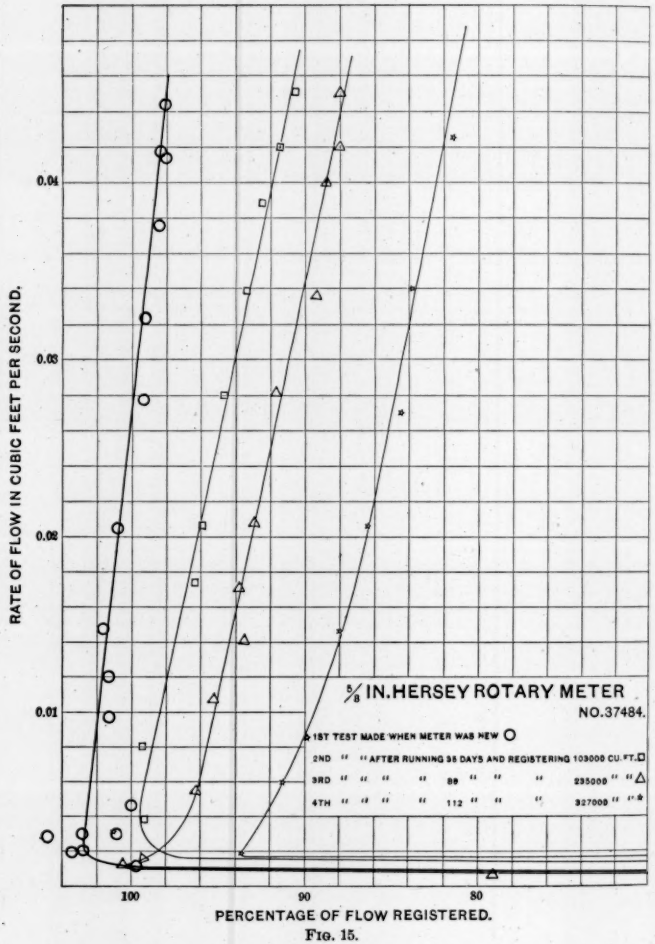

Fig. 15.

Mr. Smith

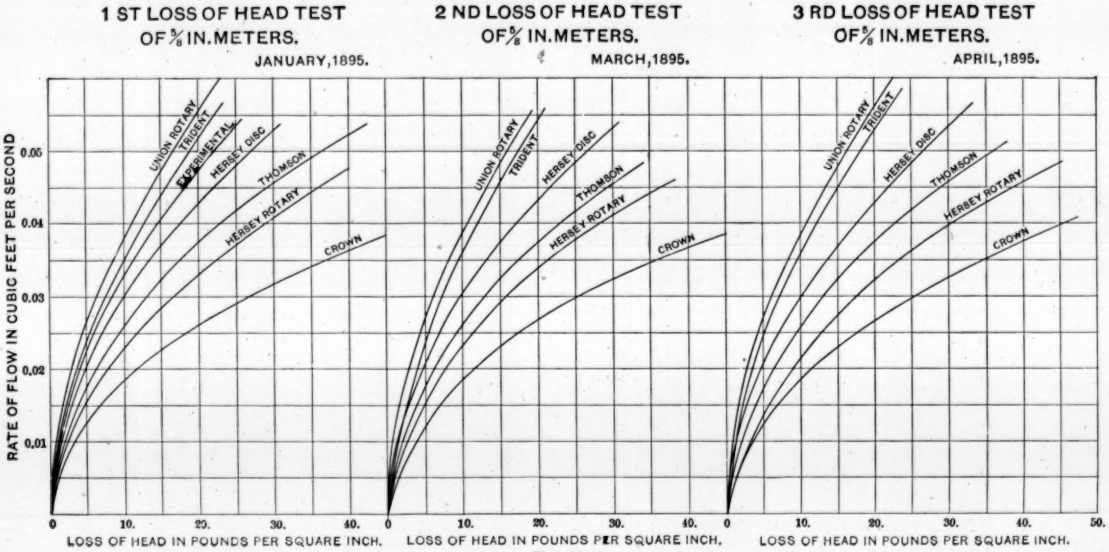

FIG. 16.

Mr. Smith. A few experiments were made to determine the effect of throttling the flow on the up-stream side of the meter, and the effect of suddenly starting and stopping the flow. These experiments indicated that neither condition materially affects the accuracy of any of the meters.

At the end of the first accuracy test, when the meters were new, the rate of flow at which the meters would just begin to register was determined. The results are shown in Table No. 23. The figures in the third and fourth columns are from the curves of the first accuracy test.

TABLE No. 23.

1	2		3		4	
Name of Meter.	Least Flow Registered by Meter.		Rate at which Meter Registers 95% of Flow.		Rate at which Meter Registers 90% of Flow.	
	Cu. ft. per second.	Gallons per minute.	Cu. ft. per second.	Gallons per minute.	Cu. ft. per second.	Gallons per minute.
Union Rotary...	0.00026	0.117	0.00075	0.337	0.00068	0.306
Experimental....	0.00024	0.108	0.00115	0.517	0.00100	0.450
Trident.....	0.00003	0.013	0.00015	0.067	0.00010	0.045
Thomson.....	0.00004	0.018	0.00058	0.261	0.00045	0.202
Hersey Disc.....	0.00013	0.058	0.00062	0.279	0.00050	0.225
Crown.....	0.00003	0.013	0.00012	0.054	0.00012	0.054
Hersey Rotary..	0.00060	0.270	0.00090	0.405	0.00085	0.382

An inspection of the experimental curves, Figs. 10 to 15, will show that the effect of wear on the three disc meters is first to increase the percentage of registration, and afterward to decrease it, while the effect on the rotary type is gradually to decrease the percentage. It will also be observed that, with the disc meters, the vertical sweeps of the registration curves are very nearly parallel to the axis of abscissas, and the horizontal sweeps to the axis of ordinates.

The first indicates the susceptibility of the meter to accurate adjustment, and the second its sensitiveness. With the Rotary meter the vertical sweep is at quite an angle with the abscissa axis, indicating that it is possible to adjust the meter to register exactly right at but one point.

It will also be observed that the curves for the disc meters are much more permanent, and that the bulk of the evidence here presented favors this type of meter. Whether this holds good for meters of larger sizes cannot now be answered.

The loss-of-head curves (Fig. 16), show that continued wear has comparatively little effect on the loss of head, and it is practically the same in all these tests.

It is to be regretted that after laboring so hard, and spending so much time, but one meter each, of seven different types, has been investigated.

Mr. Smith. The experiments illustrate very clearly the general excellence of small meters of standard patterns, and also show very forcibly the fallacy of judging the merits of meters by their initial accuracy and sensitiveness. They also illustrate how exceedingly easy it is to be deceived. For example: One of the meters which showed almost perfect registration curves, and which had a long run before failing, is now being taken out of the market by the makers and replaced by one having its good points, with the bad ones eliminated. No one can make a series of tests such as are here recorded without being impressed with the wonderful accuracy of these little instruments, and the abuse they will stand without apparent injury. He will acquire a confidence in meter measurement which unfortunately for the water-works superintendent is not shared by the average consumer.

These meters have been compared on the following points:

First.—Accuracy;
Second.—Sensitiveness;
Third.—Permanency;
Fourth.—Loss of head;
Fifth.—Capability of the meters to be adjusted for wear, and the cost of such adjustment;
Sixth.—Accessibility of the wearing parts of the meter;
Seventh.—Protection from injury from accidental freezing;
Eighth.—Cost.

Of all these, the most important is the permanency of the registration, both as regards accuracy and sensitiveness; in other words, the wearing qualities of the meter.

Instead of giving a table showing the comparison, and stating which meter is, in his opinion, the best, the speaker leaves the making of such comparison to those who are interested.

Mr. Herschel. CLEMENS HERSCHEL, M. Am. Soc. C. E.—A report of tests of water meters, unless accompanied by diagrams showing the facts that have been disclosed by the experiments, is of doubtful value, and it is to be hoped that the author will supply diagrams to illustrate his experiments. The graphic representation of such experiments is much better than any set of tables can possibly be. All the speakers have delicately alluded to the fact that the results, as given in the paper, are either obscure or misleading, and it is hoped that the author will supply the diagrams, in order that this criticism may be made invalid and that they may be published with the paper.

The experiments described and illustrated by Mr. Smith were made four years ago. At that time, or shortly after, the speaker wrote a paper in reference thereto, which was read before the New England Water-Works Association, on September 12th, 1895, and is published in the *Journal* of that Association.* The object of that paper was to argue

* *Journal* of the New England Water-Works Association, Vol. X, p. 109.

Mr. Herschel. for a public competitive test of all water meters then made. The names of the meters tested were not stated in the paper. They were referred to only by the numbers 1, 2, 3, 4, 5, 6 and 7, but their names may be given now. They are: (1) the Trident, (2) the Thomson, (3) the Hersey Disc, (4) the Union Rotary, (5) the Crown, (6) the Hersey Rotary, and (7) an experimental meter. A table in that paper gives their rank for accuracy, sensitiveness, permanency, loss of head, and price, after the first, second and third tests, as illustrated by Mr. Smith. With this explanation, the table will, of course, have more value than it had at that time. The following quotation is from the paper cited:

"They were tested for accuracy, and for loss of head caused by the meter, at various rates of discharge. Then about 100 000 cu. ft. were allowed to pass each meter, and the first set of tests were repeated. Then some 150 000 cu. ft. additional were allowed to pass each meter, and the first-named tests were again repeated. The quantity passed was equivalent to some ten years of ordinary use.

"Incidentally, therefore, these tests comprised a test for durability. Tests were also made for sensitiveness, and a test for remaining unaffected by continued disuse is still under way.

"The results were somewhat remarkable. They are given in the following table:

	First Test. Rank.							Second Test. Rank.							Third Test. Rank.						
	1	2	3	4	5	6	7	1	2	3	4	5	6	7	1	2	3	4	5	6	7
Accuracy	2	1	6	7	5	3	4	1	2	3	4	5	6	..	1	2	3	4	5	6	..
Sensitiveness	1	5	2	3	4	7	6	1	5	2	3	4	6	..	{same 1}	{same 2}	{same 5}	{same 3}	6	4	..
Permanency	..	..	..	..	..	..	..	4	1	2	3	5	6	..	1	2	4	3	5	6	..
Loss of Head	4	1	7	3	2	6	5	4	1	3	2	6	5	..	4	1	3	2	6	5	..
Price	1	{same 3}	{same 2}	6	7	4	5	..	..	..	..	..	..	..	..	..	..	..	..	..	..

The speaker was loath to give the names at the time, having tested only seven meters, and it was his desire to instigate, if he could, a public test of all water meters then made in the United States, and in the market. He still has that desire, and thinks that nothing would tend to introduce water meters more generally, provoke their use, render their use more economical and cheapen the cost of repairs, which is a serious item in all water-works economy, than to have the kinds of meters on the market reduced in number. The water-meter business to-day and for years past has been in precisely the condition in which the turbine business was about fifteen years ago. At that time the number of American makes of water wheels and turbines was legion, and every one was claimed to be better than any other. There

were probably twenty-five or thirty makes of water wheels, but by means of public competitive tests more than three-fourths of them were at once shown to be out of date, and the results of the tests were known. The water-meter purchasing public is to-day entitled to the same class of information concerning water-meters. Let any one imagine the confusion there would be in the gas business if the number of makes of gas meters were as great as the present number of makes of water-meters. It is this kind of confusion that exists to-day among water superintendents in the use of water meters. Not every water-works supertendent can afford to spend either the time or the money requisite to test all these meters in a thorough manner. The diagrams given by Mr. Smith, showing the results of the tests made four years ago may seem to be simple, but the amount of labor involved in their preparation does not appear on the surface and yet they refer to tests of only seven meters. Mr. Herschel.

The arguments made by the speaker in the paper referred to, which was addressed to a society of water-works superintendents, was that, if an association of that sort was good for anything, it should work for the common good, and, instead of relegating the testing of water meters to individual members, should appoint a committee and, for the benefit of all the members, test all meters made. That is what is needed at the present day, and the speaker hopes that the opinion of this Society, either as a Society or by its individual members, on every occasion that presents itself, will be expressed in favor of the attainment of such an object. There is plenty of precedent for it, not only the precedent stated relating to the tests of turbine wheels, but take, as an example, the testing establishment operated by the Pennsylvania Railroad Company at Altoona, Pa., where there are some twenty or thirty skilled employees, and where purchases amounting to from $15 000 000 to $20 000 000 per annum depend on the tests made by that department. That is a case where tests are conducted by a single railroad company, but that company operates thousands of miles of railroad, and is itself constituted of many subordinate companies.

Another example is that of the New England Association of Railroad Superintendents, which is constantly testing new mechanisms for the benefit of all its members, and there are other similar cases. It is economy of labor to operate in that manner, and if what the speaker has said shall tend to bring the procuring of such tests of water-meters one step nearer, he will feel amply repaid.

CORRESPONDENCE.

Mr. Harlow. JAMES H. HARLOW, M. Am. Soc. C. E.—An examination of this paper confirms the writer in the position taken by him at the American Water Works Convention, at Buffalo, in May, 1898. Table No. 1 shows that when the orifice of discharge is small, the accuracy of the meter is materially changed. Omitting from Table No. 1 the best and poorest meters, the error of registering, at $\frac{1}{32}$-in. opening, is from 10 to 32 per cent. At Buffalo the writer took the ground that in the ordinary use of water the user opens the faucet so that the discharge approximates the nominal capacity of the meter, and in such case the error is small; but it is the detection of comparatively small streams that is desirable.

From five years' records in Wilkinsburg, Pa., it is found that the average domestic consumer uses 7 500 cu. ft. of water per annum. This, at the meter rates of Wilkinsburg, would amount to $11.28. Assuming it possible that there may be an error of 10%, and that the error is in favor of the consumer, the amount actually used would be 8 300 cu. ft. This would amount to $12.40; the loss to the Water Company being $1.12. A casual examination of 3 500 meter tests shows that the errors of meters, at one-half and full capacity, about balance each other, and therefore there is no real loss. On the other hand, it is found that meters, as a rule, will allow small leaks to pass without registering, and it is for this reason that all meters used by the writer are given what is called a "time test."

This test is based on the fact that, with practically uniform pressure, the orifice of discharge is smaller when the time to pass a given quantity of water is greater. The writer's rule is: To accept no meter which passes $\frac{1}{10}$ cu. ft. of water in less than 90 seconds. In this test the valve is opened slowly and to just the amount necessary to set the meter registering, the pressure being about 130 lbs. per square inch. Under this pressure, assuming $v = 60\sqrt{2gh}$, there will be required an orifice of about 0.000044 sq. ft., which corresponds to a circular opening of a little less than $\frac{3}{32}$ in. in diameter.

Remembering that in this test the valve is opened just enough to start the meter registering, and that a less opening would allow water to pass, and, not registering, there may be a constant leak of something less than $\frac{1}{10}$ cu. ft. in 90 seconds, which is not recorded.

Assuming that 0.05 cu. ft. of water per 90 seconds does not start the meter, there is a possible leak of 17 500 cu. ft. per year, which amounts to more than twice the amount of water registered against the average customer, even after adding 10% for the assumed error.

Mr. Harlow.

The writer would like to ask if any members of the Society have records which show the relative amount registered by their consumer's meters, as compared with the total amount of water pumped. From observations made by the writer's company he is satisfied that not more than 75% of the water actually used by metered customers is measured; and he believes that in order to approximate the amount of water used there must be sought a smaller meter, or one that will register the small streams.

Since the writer began to make systematic tests of meters he has obtained an average time test of 134 seconds, as against 90 seconds obtained five years ago, and for a meter of the same make. The 90 seconds' time test gives 35 000 cu. ft. per year as a possible loss through a meter. The 134 seconds' time test, under the same conditions, gives 23 500 cu. ft., a possible gain of 12 500 cu. ft. per year, which is registered and paid for, or the leak is stopped.

The best meter now being used by the writer's company gives an average time test of 234 seconds, or at the rate of 13 500 cu. ft. of water per year.

In June, 1898, four of the best piston meters were placed tandem on the service line furnishing water to the building in which the Water Company has its office. These meters were tested at the time they were placed on the service line, and have been again tested to-day.

From Saturday afternoon, June 4th, until Monday morning, June 6th, 1898, for 38 hours 30 minutes these four meters were tested by allowing as small a stream of water to pass as would just move the register of the slowest meter. At the end of this time, meter No. 1 read 58 cu. ft.; No. 2, 17 cu. ft.; No. 3, 19 cu. ft.; and No. 4, 42 cu. ft. Assuming the efficiency of meter No. 1 to be equal to unity, then No. 2 will equal 0.293; No. 3, 0.344, and No. 4, 0.724. The total amount of water passing these four meters in eight months was: No. 1, 26 065 cu. ft.; No. 2, 17 738 cu. ft.; No. 3, 20 043 cu. ft., and No. 4, 25 407 cu. ft. Assuming No. 1 to be correct, their ratios of efficiency will be: No. 2, 0.671; No. 3, 0.768, and No. 4, 0.985.

Comparing the above figures with the tests given in Table No. 24, it will be seen that when the meters were placed on the service line the error in one-half and full stream was within $\frac{7}{10}$ of 1%, which is the record of meter No. 3.

It will also be noted that, with the exception of meter No. 2, all these meters, under the test made to-day, register from 1 to 5% slow.

The meter tested under the longest time test shows the largest amount of water passing during the eight months. The difference as shown is due to small leaks which meters Nos. 1 and 2 did not succeed in registering.

Mr. Harlow. The writer will be pleased if someone else takes up this question of a meter that will catch the small streams.

TABLE No. 24.

METER.	No. 1.		No. 2.		No. 3.		No. 4.	
Date of test.	June, 1898.	Jan., 1899.	June, 1898.	Jan., 1899.	June, 1898.	Jan., 1899.	June, 1898.	Jan., 1899.
	Secs.	Secs.	Secs.	Secs.	Secs.	Secs.	Secs.	Secs.
Time test.	1 121	228	150	226	119	107	261	222
Half open.	O. K.	1% slow.	O. K.	O. K.	0.7% slow.	1.8% slow.	O. K.	4% slow.
Full open.	O. K.	1.3% "	O. K.	O. K.	0.7% "	1.7% "	O. K.	5% "

Mr. Hawley. W. C. HAWLEY, Assoc. M. Am. Soc. C. E.—A careful investigation of the subject has led the writer to the same conclusion as that reached by the author regarding the relative merits of the rotary and disc types of water meters. The disc meter, for a given size, costs less, and while, when new, there is little difference in accuracy and sensitiveness between the two types, after having registered 100 000 cu. ft., or more, the advantage as a rule is decidedly with the disc meters.

Another point in favor of the disc meters is that sand or other foreign matter is less likely to stop them than it is to stop the rotary meters. This is especially true in the smaller sizes, and where the water pressure is not high. This is an important matter, for rotary meters of the Crown and Hersey types will usually, when stopped, pass from 60 to 75% of the full flow when running, and hence the stoppage does not attract attention, and frequently the loss in registration amounts to more than the cost of the meter.

Illustrating the surprising accuracy of new disc meters mentioned by the author, the Water Department of which the writer is Superintendent recently purchased 1 000 disc meters. The specifications for $\frac{5}{8}$-in. and $\frac{3}{4}$-in. meters required that the average of the tests on the larger streams, down to and including the tests on the $\frac{1}{2}$-in. stream, should be within the limits of 625 and 630 lbs. on a registration of 10 cu. ft., and that no single test should exceed the limits of + 1% and — 2% (618 $\frac{3}{4}$ lbs. and 637 $\frac{1}{2}$ lbs.). The $\frac{1}{16}$-in. and $\frac{1}{32}$-in. tests should be within the limits of + 1% and — 5% (61$\frac{9}{10}$ lbs. and 65$\frac{2}{3}$ lbs.) on a registration of 1 cu. ft. The tests for 1-in. meters were the same, except that no test on a 3$\frac{1}{2}$-in. stream was required. Of the first 250 meters delivered, about sixty failed to register within the limits, though previously tested satisfactorily at the factory. On investigation it was found that water at a temperature of about 50° Fahrenheit had been used for testing at the factory, while the water

used in testing for the Water Department, had a temperature of about 70 degrees. So closely does the disc fit the disc chamber, that this slight difference in temperature affected the registration. A little scraping of the discs brought all the rejected meters within the requirements. Of the balance of the order, 750, not one failed to test satisfactorily. Mr. Hawley.

The author considers the essentials of a good water meter to be, as quoted from Mr. Iben:

(1) Sensitiveness or accuracy of registry at different rates of discharge;

(2) Capacity for a given size of meters;

(3) Cost;

(4) Durability.

The writer would suggest the propriety of placing accuracy and sensitiveness under separate heads. A meter may be very accurate on high rates of discharge and fail absolutely to register a small rate; or it may be sensitive to both large and small rates and accurate on but part or none of them. It is desirable that a meter should be sensitive to any flow, and as accurate within proper limits as good substantial construction and reasonable cost will allow. A meter made so sensitive that it will register on the extremely small rates of discharge will probably be too delicate to stand the severe strain of measuring at high rates of discharge. Just what degree of sensitiveness should be required on small rates of discharge must be determined by the conditions under which the meter works, the pressure, amount of silt and sand in the water, etc. For the $\frac{5}{8}$-in. and $\frac{3}{4}$-in. sizes a rate of 1 cu. ft. per hour should be the limit required for new meters under ordinary conditions, though they will frequently do even better.

To the essentials for a good water meter which are mentioned by the author, the writer would add:

(*a*) Accessibility of working parts, and ease and speed with which they can be reassembled;

(*b*) A minimum of cost of repairs, in case of damage caused by freezing;

(*c*) A minimum of liability to stoppage from sand or other foreign substance.

These relate directly to the cost of maintenance, and, in the course of the life of a meter, are important items.

In asking for proposals for furnishing meters in August, 1897, the writer specified that each bidder should submit with his proposal the following guarantees:

"A definite guarantee must be given by each bidder for each size of meter, specifying the maximum cost of repairing meters which have been damaged by freezing. Said guarantee to cover the cost of new parts to replace those damaged by frost, and also the time of man in making the repairs; and this guarantee shall remain in force

Mr. Hawley. so long as the meters purchased under these specifications are in service.

"A similar guarantee must be given specifying the maximum cost of repairing meters of the various sizes in case of damage by hot water.

"A similar guarantee must be given specifying for each size of meter the cost of such principal parts as may be required for renewing worn parts or for repairing meters accidentally damaged."

In the above guarantees "the time of man in making repairs" was included because of the inaccessibility of the working parts of some meters, and the consequent loss of time in making repairs as compared with others. The representative of each meter company bidding was ready, in conversation, with sundry proofs and reasons why his particular meter was the most economical to maintain; but when called upon to state this in actual figures with a bond behind them, only one of four complied. The advisability of exacting these guarantees, particularly that relating to damage by freezing, has been fully demonstrated during the cold weather of the present winter.

A year ago the writer made tests of some meters that had been in service for about twenty months, and had passed large quantities of water. Table No. 25 shows both the original test and the second test, and compares the two. The percentages of registration given are the ratios of the meter registration to the tank measurement in each case, *i. e.*,

$$100\left(\frac{\text{meter registration}}{\text{tank measurement}}\right).$$

The writer prefers this to the author's method, as the ratio is based upon exact measurement, and not upon the constantly varying registration of the meter. Hence it gives at once just the percentage of water actually passed by the meter and registered, and avoids the necessity of using the + and — signs. These tests were made under a pressure of 38 to 43 lbs., the water being brought to the testing table in a 4-in. pipe from a point on the force main close to the stand pipe. The pressure was therefore very uniform. Its temperature, though not taken, was known to vary but slightly.

After testing, the meters were taken apart and examined carefully, and their condition noted.*

These and other tests and examinations lead the writer to believe that while most of the standard makes of disc meters are reliable for ordinary service when new, there is a decided difference in their durability, and hence some of them are much less sensitive than others after having been in service for a time.

The principal reason for this is the friction between the disc and the vertical diaphragm which divides the disc chamber. In several styles of disc meters the tendency of the disc to rotate about a line coinciding

* See the column headed Remarks.

with the center of its spindle, is overcome by direct contact between the edge of the disc and the diaphragm. This constant rubbing, aided by what little grit there may be in the water, in course of a few million oscillations of the disc causes wear, both on the diaphragm, and on the edge of the disc in contact with it, and a consequent increase in friction to such an extent that the meter loses, to a greater or less degree, its sensitiveness to small flows. This is frequently so serious a matter that nothing smaller than a $\frac{1}{8}$-in. stream will be registered. This means that in case of a small leak, from 500 to over 4 000 galls. per 24 hours would pass without registering. One or two manufacturers have overcome this difficulty by introducing a roller which is attached to the edge of the disc and which works either in a slot cut into the inner surface of the disc chamber, or directly against the diaphragm itself. This prevents the sliding contact between disc and diaphragm and the consequent wear. Mr. Hawley.

Incidentally, the writer would call attention to the fact that the lack of sensitiveness, after having been in service for a time, is more marked in rotary than in disc meters, and doubtless accounts to a large extent for the considerable difference between total pumpage and total meter registration, which has been noticed in cities where such records have been kept, and which has been mentioned in papers and discussions before this Society.

The intermediate gear train which transfers and reduces the motion from the disc spindle to the dial clockwork is one of the delicate parts of a meter, and wear here soon affects the sensitiveness of the meter. The use of a differential gear for this purpose has proven very unsatisfactory. Some of the forms of single-train gears are little better, though there are some excellent ones in the market. One of the best gear trains made, both as regards a minimum of friction and wearing qualities, is the compound train in the Trident meter, which was designed by John Thomson, M. Am. Soc. C. E.

One bad form of construction, sometimes used, is to fasten a gear to a shaft with a set-screw. With the continual jar due to the operation of the meter the set-screw is very liable to work loose or slip on the shaft and hence the meter fails to register.

The writer doubts the practicability of a manufacturer's diagram to indicate the proper correction for registration as time passes, which is based upon the quantity registered only. Time is an important element in the wear of a meter. A given quantity of water measured at a high rate of discharge will cause much more wear than the same quantity measured at a low rate. An actual test is probably the only safe method of determining the matter.

Mr. Hawley.

TABLE No. 25—(*Continued*).—HERSEY DISC METERS TESTED IN FEBRUARY, 1898.

5/8..	47 208	3/4	625	641½	20	151 000	625	666½	97.4	93.8	— 3.6	Good condition.
	.....	1/4	62½	63	.....	.....	62½	66	99.2	94.7	— 4.5	
	.....	1/8	62½	63	.....	.....	62½	67	99.2	93.3	— 5.9	
	.....	1/16	31¼	32	.....	.....	Would not	register.	97.7	0.0	—97.7	
5/8..	46 065	3/4	625	631	22	146 000	625	628½	99.0	99.4	+ 0.4	Good condition.
	.....	1/4	62½	61	.....	.....	62½	62	102.4	100.8	— 1.6	
	.....	1/8	62½	62½	.....	.....	62½	63½	100.0	98.8	— 1.2	
	.....	1/16	31¼	33	.....	.....	31¼	37	94.7	84.5	—10.2	
	.....	1/32	Not	tested.	.....	.....	Regis	tered.	.....	.....	.....	
5/8..	46 063	3/4	625	635	22	112 000	625	636½	98.4	98.2	— 0.2	Good condition except guide frame for spindle.
	.....	1/4	62½	61½	.....	.....	62½	62½	101.6	100.0	— 1.6	
	.....	1/8	62½	61½	.....	.....	62½	63	101.6	99.2	— 2.4	
	.....	1/16	31¼	31½	.....	.....	31¼	34	99.2	91.9	— 7.3	
	.....	1/32	Not	tested.	.....	.....	Regis	tered.	.....	.....	.....	
5/8..	46 120	3/4	625	632	22	177 000	625	688½	98.9	90.8	— 8.1	Diaphragm somewhat worn, also disc chamber. On 1/16-in. stream meter ran 3½ tenths of cu. ft. and stopped. Would not register again on 1/16-in. stream.
	.....	1/4	62½	63	.....	.....	62½	74	99.2	84.5	—14.7	
	.....	1/8	62½	61	.....	.....	62½	71½	102.4	87.4	—15.0	
	.....	1/16	31¼	32½	.....	.....	21 9/16	40	96.2	54.8	—41.4	
	.....	1/32	Not	tested.	.....	.....	Would not	register.	.....	.....	.....	
3/4..	46 055	3/4	625	633	23	125 300	625	656½	98.7	95.2	— 3.5	Disc chamber worn bright on one side. Diaphragm considerably worn. Clips for holding guide frame for spindle were worn off. Frame loose.
	.....	1/4	62½	62¾	.....	.....	62½	66½	99.6	94.0	— 5.6	
	.....	1/8	62½	61¼	.....	.....	62½	67	102.0	93.3	— 8.7	
	.....	1/16	31¼	31½	.....	.....	Would not	register.	99.2	0.0	—99.2	
	.....	1/32	6¼	.....	.....	.....	.....	.....	.....	.....	.....	
3/4..	46 047	3/4	625	626½	23	137 500	625	655	99.8	95.4	— 4.4	Diaphragm slightly worn, also disc chamber. Clips for holding guide frame for spindle in place were worn off. Frame loose.
	.....	1/4	62½	61¾	.....	.....	62½	65	101.2	96.2	— 5.0	
	.....	1/8	62½	62	.....	.....	62½	65	100.8	96.2	— 4.6	
	.....	1/16	31¼	32	.....	.....	31¼	36½	97.7	85.3	—12.4	
	.....	1/32	6¼	6¼	.....	.....	Regis	tered.	100.0	?	.....	
1..	46 032	1	Not	tested.	23	100 529	625	637	.....	98.1	.....	
	.....	3/4	625	634	.....	.....	625	636½	98.5	98.2	— 0.3	Disc spindle worn on one side. Guide for spindle showed a little wear.
	.....	1/2	Not	tested.	.....	.....	625	635	.....	98.4	.....	
	.....	1/4	62½	62¼	.....	.....	62½	62½	100.4	100.0	— 0.4	
	.....	1/8	62½	62½	.....	.....	62½	63	100.0	99.2	— 0.8	
	.....	1/16	31¼	33⅞	.....	.....	31¼	35	93.3	89.3	— 4.0	
	.....	1/32	6¼	6¾	.....	.....	Regis	tered.	92.6	?	.....	

Mr. Hawley.

TABLE No. 25.—BEE METERS TESTED IN FEBRUARY, 1898.

Size. Inches.	No. of meter.	Size of stream. Inches.	Original Test. Meter. Pounds.	Original Test. Scales. Pounds.	Time in service. Months.	Total registration. Cubic feet.	Second Test. Meter. Pounds.	Second Test. Scales. Pounds.	Registration. Original test. Percentage.	Registration. Second test. Percentage.	Registration. Change in registration. Percentage.	Remarks.
⅝..	75 100	¾	625	630	20	119 000	625	630	99.2	99.2	0.0	Meter in good condition, except diaphragm and disc, which were both somewhat worn. Differential gear (intermediate) not examined.
.....	.....	½	62½	62½	.....	.....	62½	63	100.0	99.2	— 0.8	
.....	.....	⅜	62½	63½	.....	.....	62½	65	98.4	96.2	— 2.2	
.....	.....	¼	31¼	32½	.....	.....	31¼	39	96.2	80.1	— 16.1	
.....	.....	3/32	Not tested.		.....	.....	Meter regist'd.		.....	?	.....	
⅝..	73 581	¾	625	629	23	233 000	625	665½	99.4	94.0	— 5.4	Differential gear badly worn. Diaphragm and disc considerably worn. Disc chamber slightly worn.
.....	.....	½	62½	63	.....	.....	62½	66½	99.2	94.3	— 4.9	
.....	.....	⅜	62½	63½	.....	.....	62½	69½	98.4	90.0	— 8.4	
.....	.....	¼	31¼	33	.....	.....	Did not register.		94.7	0.0	— 94.7	
.....	.....	3/32	6¼	6¼	.....	.....	.....	.....	100.0	0.0	—100.0	
⅝..	75 080	¾	625	625½	21	98 900	625	637	99.9	98.1	— 1.8	Differential gear somewhat worn, also diaphragm.
.....	.....	½	62½	61	.....	.....	62½	62½	102.4	100.0	— 2.4	
.....	.....	⅜	62½	63½	.....	.....	62½	64	98.4	97.7	— 0.7	
.....	.....	¼	31¼	33½	.....	.....	Did not register.		93.3	0.0	— 93.3	
¾..	73 668	¾	625	633	23	457 800	625	649½	98.7	96.2	— 2.5	Differential gear badly worn.
.....	.....	½	62½	62½	.....	.....	62½	66	100.0	94.7	— 5.3	Diaphragm considerably worn.
.....	.....	⅜	62½	64	.....	.....	62½	68	97.7	91.9	— 5.8	
.....	.....	¼	31¼	32½	.....	.....	Did not register.		95.4	0.0	— 95.4	
.....	.....	3/32	6¼	7½	.....	.....	.....	.....	83.3	0.0	— 83.3	
¾..	81 914	¾	625	623	19	110 000	625	628	100.3	99.5	— 0.8	Differential gear somewhat worn, also diaphragm.
.....	.....	½	62½	60½	.....	.....	62½	63	103.3	99.2	— 4.1	
.....	.....	⅜	62½	62½	.....	.....	62½	64½	100.0	96.9	— 3.1	
.....	.....	¼	31¼	33	.....	.....	31¼	37½	94.7	83.3	— 11.4	
.....	.....	3/32	Not tested.		.....	.....	Regis tered.		.....	?	.....	
.....	.....	1	"	"	.....	.....	625	668	.....	93.6	.....	
1..	73 683	¾	625	632	23	537 270	625	671	98.9	93.1	— 5.8	Disc chamber worn bright.
.....	.....	½	625	626½	.....	.....	625	667	99.9	93.7	— 6.2	Diaphragm somewhat worn.
.....	.....	⅜	62½	62½	.....	.....	62½	79	100.0	79.1	— 20.9	No record made of condition of differential gear.
.....	.....	¼	62½	65	.....	.....	Did not register.		96.2	0.0	— 96.2	
.....	.....	3/32	31¼	32½	.....	.....	"	"	96.2	0.0	— 96.2	

TABLE No. 25—(*Continued*).—NASH METERS TESTED IN FEBRUARY, 1898.

5..	158 826	3/4	625	628½	23	160 000	625	670	99.4	93.3	— 6.1	Spindle worn. Diaphragm and disc chamber slightly worn, otherwise in good condition.
	.....	1/2	62½	62½	.....	.....	62½	67	100.0	93.3	— 6.7	
	.....	1/4	62½	62½	.....	.....	62½	67	100.0	93.3	— 6.7	
	.....	1/8	31½	31½	.....	.....	31½	37¼	98.4	88.9	- 14.5	
	.....	1/16	6½	6½	.....	.....	Regis tered.		100.0	?	.....	
5..	158 795	3/4	625	626½	24	117 700	625	662	99.8	94.4	— 4.4	Spindle badly worn. Diaphragm worn, otherwise in good condition.
	.....	1/2	62½	62½	.....	.....	62½	66¼	100.0	94.0	— 6.0	
	.....	1/4	62½	62½	.....	.....	62½	66	100.4	94.7	— 5.7	
	.....	1/8	31½	31½	.....	.....	31½	36	99.2	86.8	— 12.4	
	.....	1/16	6½	6½	.....	.....	Would not register.		100.0	0.0	.....	
5..	158 820	3/4	625	628½	24	108 600	625	661	99.4	94.6	— 4.8	Spindle badly worn. Diaphragm slightly worn, otherwise in good condition.
	.....	1/2	62½	62½	.....	.....	62½	66	100.0	94.7	— 5.3	
	.....	1/4	62½	62	.....	.....	62½	68	100.8	91.9	— 8.9	
	.....	1/8	31½	31	.....	.....	Would not register.		100.8	0.0	—100.8	
	.....	1/16	6½	6½	.....	.....	"	"	100.0	.....	.....	
3..	160 538	3/4	625	622	22	124 400	625	620	100.5	100.8	+ 0.3	Spindle nearly worn off and driving arm badly worn.
	.....	1/2	No test.		.....	.....	62½	62½	.....	100.0	.....	
	.....	1/4	62½	64½	.....	.....	62½	61½	96.9	101.6	+ 4.7	Diaphragm slightly worn, otherwise in good condition.
	.....	1/8	62½	61½	.....	.....	62½	61½	101.6	101.2	— 0.4	
	.....	1/16	31½	31½	.....	.....	62½	62½	99.2	100.0	+ 0.8	
	.....	1/32	No test.		.....	.....	Regis tered.		.....	.....	.....	
3..	158 732	3/4	625	625	24	178 600	625	642½	100.0	97.3	— 2.7	Disc chamber somewhat worn. Spindle slightly worn. Diaphragm considerably worn.
	.....	1/2	62½	60?	.....	.....	62½	63	102.9	99.2	— 3.7	
	.....	1/4	62½	59	.....	.....	62½	62½	106.0	100.0	— 6.0	
	.....	1/8	31½	29½	.....	.....	Would not register.		106.0	0.0	—106.0	
	.....	1/16	6½	6½	.....	.....	"	"	96.2	.....	.....	
3..	160 544	3/4	625	630½	22	235 100	625	660	99.1	94.7	— 4.4	Meter found stuck and disc warped out of shape, probably by hot water. Put in new disc and tested, with results shown. Spindle of old disc badly worn, also diaphragm. Spindle considerably worn. Diaphragm slightly worn. Otherwise meter in good condition.
	.....	1/2	62½	61½	.....	.....	62½	65	101.6	96.2	— 5.4	
	.....	1/4	62½	60	.....	.....	62½	64½	104.2	96.9	— 7.3	
	.....	1/8	31½	31½	.....	.....	Would not register.		98.4	0.0	— 98.4	
	.....	1/16	6½	6½	.....	.....	"	"	96.2	.....	.....	
3..	160 531	3/4	625	624½	22	235 600	625	644	100.1	97.0	— 3.1	
	.....	1/2	62½	59½	.....	.....	62½	65	105.0	96.2	— 8.8	
	.....	1/4	62½	63½	.....	.....	62½	63	98.4	99.2	+ 0.8	
	.....	1/8	31½	31½	.....	.....	62½	65	98.4	96.2	— 2.2	
	.....	1/16	No test.		.....	.....	Would not register.		.....	.....	.....	
	.....	1/32										
1..	158 888	1	625	632½	20	705 380	625	660	98.8	94.7	— 4.1	Disc and spindle worn. Disc chamber worn. Rest of meter in good condition.
	.....	3/4	625	624½	.....	.....	625	665	100.1	94.0	— 6.1	
	.....	1/2	62½	65	.....	.....	62½	78	96.2	80.0	— 16.2	
	.....	1/4	62½	58½	.....	.....	62½	67	106.8	93.3	— 13.5	
	.....	1/8	31½	32	.....	.....	62½	78	97.7	80.0	— 17.7	
	.....	1/16	No test.		.....	.....	Regis tered.		.....	.....	.....	
	.....	1/32										

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TABLE No. 25—(*Continued*).—NIAGARA METERS TESTED IN FEBRUARY, 1898.

Size. Inches.	No. of meter.	Size of stream. Inches.	Original Test. Meter. Pounds	Original Test. Scales. Pounds	Time in service. Months.	Total registration. Cubic feet.	Second Test. Meter. Pounds.	Second Test. Scales. Pounds.	Registration. Original test. Percentage.	Registration. Second test. Percentage.	Registration. Change in registration. Percentage.	Remarks.
3	5 355	3/4	625	629½	23	88 800	625	639	99.2	97.8	−1.4	Intermediate gear considerably worn. Diaphragm slightly worn. Disc good.
	.....	1/2	62½	61½	.....	.....	62½	63½	101.2	98.4	−2.8	
	.....	3/8	62½	61½	.....	.....	62½	64	101.2	97.7	−3.5	
	.....	1/4	31¼	31	.....	.....	31¼	36½	100.8	85.6	−15.2	
	.....	1/8	Not tested.		.....	.....	Regis	tered.	?	0.0	.....	
	.....	1/16			.....	.....						
3	5 308	3/4	625	631	23	147 800	625	648	99.6	96.5	−2.5	Intermediate gear somewhat worn. Spindle worn. Diaphragm very slightly worn.
	.....	1/2	62½	61	.....	.....	62½	64½	102.4	96.9	−5.5	
	.....	3/8	62½	61	.....	.....	62½	65	102.4	96.2	−6.2	
	.....	1/4	31¼	31¼	.....	.....	31¼	37½	100.0	83.3	−16.7	
	.....	1/8	Not tested.		.....	.....	Did not	register.	?	0.0	.....	
	.....	1/16			.....	.....						
3	5 396	3/4	625	630½	22	69 500	625	640	99.1	97.7	−1.4	Diaphragm very slightly worn. Otherwise meter in good condition.
	.....	1/2	62½	62	.....	.....	62½	63	100.8	99.2	−1.6	
	.....	3/8	62½	61½	.....	.....	62½	64½	102.0	96.9	−5.1	
	.....	1/4	31¼	32½	.....	.....	Did not	register.	96.2	0.0	−96.2	
	.....	1/8			.....	.....						
3	5 951	3/4	625	629	21	138 100	625	622½	99.4	100.4	+1.0	Diaphragm badly worn, also edges of disc. Otherwise in good condition.
	.....	1/2	62½	61½	.....	.....	62½	61½	101.2	101.2	0.0	
	.....	3/8	62½	61	.....	.....	62½	62½	102.4	100.0	−2.4	
	.....	1/4	31¼	29½	.....	.....	Did not	register.	105.9	0.0	−105.9	
	.....	1/8			.....	.....						
3	6 024	3/4	625	624	20	216 100	625	614	100.1	101.8	+1.7	Spindle somewhat worn, otherwise in good condition.
	.....	1/2	62½	60	.....	.....	62½	61	104.2	102.4	−1.8	
	.....	3/8	62½	59½	.....	.....	62½	60½	105.0	103.3	−1.7	
	.....	1/4	31¼	30½	.....	.....	31¼	33½	102.4	92.6	−9.8	
	.....	1/8	Not tested.		.....	.....	Regis	tered.	?	?	.....	
	.....	1/16			.....	.....						
3	5 973	3/4	625	630	21½	172 300	625	610	99.2	102.4	+3.2	Diaphragm slightly worn, otherwise in good condition.
	.....	1/2	62½	62½	.....	.....	62½	61	100.0	102.4	+2.4	
	.....	3/8	62½	60	.....	.....	62½	61½	104.2	101.6	−2.6	
	.....	1/4	31¼	30½	.....	.....	31¼	32½	101.6	96.2	−5.4	
	.....	1/8	Not tested.		.....	.....	Did not	register.	?	0.0	.....	
	.....	1/16			.....	.....						
3	5 950	3/4	625	625½	21	235 200	625	625	99.9	100.0	+0.1	Meter in fine condition. Wear on disc chamber perfectly even. The whole interior surface of disc chamber polished bright.
	.....	1/2	62½	62½	.....	.....	62½	62	100.0	100.8	+0.8	
	.....	3/8	62½	62½	.....	.....	62½	63	100.0	99.2	−0.8	
	.....	1/4	31¼	31¼	.....	.....	31¼	33	99.2	94.7	−4.5	
	.....	1/8	Not tested.		.....	.....	Meter re	gistered.	?	?	.....	
	.....	1/16			.....	.....						

TABLE No. 25.—TRIDENT METERS TESTED IN FEBRUARY, 1898.—(*Continued.*)

¾..	17 397	¾	62½	61½	22	168 600	625	615½	101.6	101.5	− 0.1	Meter in good condition. No wear apparent.
.....	.....	½	62½	61	.....	.....	62½	61	102.4	102.4	0.0	
.....	.....	¼	62½	60½	.....	.....	62½	60½	103.3	103.7	+ 0.4	
.....	.....	Y8	31¼	31	.....	.....	31½	31½	100.8	100.0	− 0.8	
.....	.....	32	6½	7	.....	.....	6½	6½	89.3	100.0	+10.7	
¾..	17 387	¾	62½	63	22	149 500	625	638½	99.2	97.9	− 1.3	Meter in good condition. No wear apparent.
.....	.....	½	62½	62½	.....	.....	62½	63½	100.0	98.4	− 1.6	
.....	.....	¼	62½	62¼	.....	.....	62½	63½	100.0	98.4	− 1.6	
.....	.....	Y8	62½	65	.....	.....	31½	38	96.2	82.2	−14.0	
.....	.....	32	Regis	tered.	.....	.....	Regis	tered.	?	?	.....	
¾..	17 393	¾	625	635	22	149 200	625	622½	98.4	100.4	+ 2.0	Meter in good condition. No wear apparent, except that change gear had slipped off of driving shaft from intermediate gear, and end of shaft was slightly worn. Change gear replaced and meter tested, with results shown.
.....	.....	½	62½	62	.....	.....	62½	62½	100.8	100.4	− 0.4	
.....	.....	¼	62½	62½	.....	.....	62½	62½	100.0	100.0	0.0	
.....	.....	Y8	31¼	32½	.....	.....	31½	35½	96.2	88.0	− 8.2	
.....	.....	32	Not	tested.	.....	.....	Regis	tered.	.....	?	.....	
¾..	17 390	¾	625	620	22	202 000	62½	64½	99.4	97.3	− 2.1	This meter had been working entirely immersed in muddy water most of time since first set. First test on ¾-in. stream gave 750 lbs. for 10 cu. ft. registration. Examination showed that grit had entered stuffing box of driving shaft from intermediate gear, causing it to turn very hard, and the intermediate gear was worn out. Replaced with new one and tested, as shown.
.....	.....	½	62½	62½	.....	.....	62½	63½	100.0	98.4	− 1.6	
.....	.....	¼	62½	63	.....	.....	62½	62½	99.2	100.0	+ 0.8	
.....	.....	Y8	31½	31½	.....	.....	31½	33	99.2	94.7	− 4.5	
.....	.....	32	Not	tested.	.....	.....	Regis	tered.	?	?	.....	

TABLE No. 25—(*Continued*).—TRIDENT METERS TESTED IN FEBRUARY, 1898.

Size. Inches.	No. of meter.	Size of stream. Inches.	Original Test. Meter. Pounds	Original Test. Scales. Pounds	Time in service. Months.	Total registration. Cubic feet.	Second Test. Meter. Pounds.	Second Test. Scales. Pounds.	Registration. Original test. Percentage.	Registration. Second test. Percentage.	Registration. Change in registration. Percentage.	Remarks.
5/8 ..	17 262	3/4	625	630	22	111 400	625	640	99.2	97.7	— 1.5	In good condition. When first tested was not as sensitive as should have been. Examination showed an eel-skin in disc chamber around disc, but no damage done. Skin was removed, meter reassembled and tested, with result given.
	.....	1	62½	62½	.....	.....	62½	63	100.0	99.2	— 0.8	
	.....	1 1/4	62½	63	.....	.....	62½	63½	99.2	98.4	— 0.8	
	.....	1 1/2	31¼	40	.....	.....	31¼	36	78.1	86.8	+ 8.7	
	.....	1 5/8	Not tested.		.....	.....	Regis tered.		.....	.....	.....	
	.....	1 3/4										
5/8 ..	17 280	3/4	625	624	22	115 000	625	652	100.2	95.9	— 4.3	Meter in good condition. No wear apparent.
	.....	1	62½	61½	.....	.....	62½	65½	101.6	95.4	+ 6.2	
	.....	1 1/4	62½	62½	.....	.....	62½	67	100.0	93.3	— 6.7	
	.....	1 1/2	31¼	33½	.....	.....	31¼	39	93.3	80.1	—13.2	
	.....	1 5/8	Not tested.		.....	.....	Regis tered.		.....	.....	.....	
	.....	1 3/4										
5/8 ..	17 366	3/4	625	623	23	101 000	625	624	100.3	100.2	— 0.1	Meter in good condition. No wear apparent.
	.....	1	62½	62	.....	.....	62½	62½	100.8	100.4	— 0.4	
	.....	1 1/4	62½	63½	.....	.....	62½	62½	98.4	100.0	+ 1.6	
	.....	1 1/2	31¼	33½	.....	.....	31¼	32	94.0	97.7	+ 3.7	
	.....	1 5/8	Not tested.		.....	.....	250	250	?	100.0	.....	
	.....	1 3/4										
3/4 ..	17 384	3/4	62½	62½	24	124 000	625	636	100.0	98.3	— 1.7	Meter in good condition. No wear apparent.
	.....	1	62½	62½	.....	.....	62½	63	100.0	99.2	— 0.8	
	.....	1 1/4	62½	62½	.....	.....	62½	64	100.0	97.7	— 2.3	
	.....	1 1/2	62½	63½	.....	.....	62½	64½	98.8	96.9	— 1.9	
	.....	1 5/8	Not tested.		.....	.....	62½	65½	?	95.8	.....	
	.....	1 3/4										
3/4 ..	17 394	3/4	62½	62½	22	132 000	625	642	100.0	97.4	— 2.6	Meter in good condition. No wear apparent.
	.....	1	62½	62½	.....	.....	62½	64	100.4	97.7	— 2.7	
	.....	1 1/4	62½	61½	.....	.....	62½	64	101.6	97.7	— 3.9	
	.....	1 1/2	31¼	31½	.....	.....	6¼	6¾	100.0	92.6	— 7.4	
	.....	1 5/8	6¼	7	.....	.....	Regis tered.		89.3	?	.....	
	.....	1 3/4										

Mr. Kuichling. E. KUICHLING, M. Am. Soc. C. E.—This paper is a valuable addition to the literature of a subject which is of great importance in all large communities, and which has caused much worry to those who have charge of public water supplies. Many interesting experiments are set forth by the author, and the conclusions which he has deduced therefrom are presented so clearly and forcibly that little remains to be added in the discussion. Some of the topics, however, leave room for amplification, and in the following an attempt is made to bring to special notice the question of the permissible limit of error in registration at low rates of discharge; also to indicate the durability of meters by giving the results of a recent examination of a large number of small meters which have remained in use for long periods of time.

Taking conditions as they actually exist for the usual small meters in dwellings, boarding houses, etc., where water is not used for manufacturing purposes, it can be said that not more than 300 galls. per day are consumed by an average family of five persons, and that such use almost invariably occurs at the greatest practicable rate of delivery, or with the faucet wide open. With small service pipes and moderate pressures in the street mains, a maximum discharge of 200 galls. per hour through the meter may be assumed, so that for a consumption of 300 galls. per day in an ordinary dwelling, having all its pipes and fixtures in perfect order, the meter is in operation only about $1\frac{1}{2}$ hours, and is consequently idle about $22\frac{1}{2}$ hours each day. If, however, the pipes or fixtures in such a dwelling are defective, the leakage during the latter period of time will necessarily pass through the meter, and unless the same is very sensitive, much of this discharge may escape without registration.

The writer will now consider the magnitude of such leakage under ordinary conditions of pressure and public conscience. Usually the hot-water faucets are the first to commence dribbling, and next in order come the ball-cocks of water-closet tanks. By actual measurement of many such dribblings, which were regarded as insignificant by the householders, the writer has found a discharge of from 3 to 6 galls. per hour, or from 72 to 144 galls. per day, from a single faucet or ball-cock; and in many other cases where a dwelling was supplied from an attic tank of from 100 to 150 galls. capacity, and with the pipes and fixtures apparently in fair order, it was found that when the pressure was reduced soon after midnight to such extent as to prevent water from entering the tank through the open ball-cock, there would be little water available for use 6 or 8 hours afterward, notwithstanding that none of the fixtures had been used during the night. From these experiences, the writer has reached the conclusion that the leakage from defective pipes and fixtures in an average dwelling, during the aggregate period of $22\frac{1}{2}$ hours of each day when no

faucet is opened by any of the occupants, may easily become as large as the legitimate consumption before it attracts the notice of the householder. It therefore follows that if the advantage of a meter system is to be even approximately realized, the meters must be sensitive enough to record a reasonable percentage of a leakage which amounts to much in the course of 24 hours, but occurs at so small a rate of flow as to appear insignificant in comparison with the discharge of a wide open faucet. Mr. Kuichling.

The question now arises as to the requisite degree of sensitiveness or registration of the meter under these low discharges. In view of the fact that the magnitude of the leakage may easily reach 25% of the entire daily delivery of the water-works system, it would appear at first glance that the meter should exhibit, when passing water at the rate of about 72 galls. in 24 hours, or 3 galls. per hour, a registration of at least 50% of this quantity, so as to divide the loss equally between the consumer and the corporation furnishing the water; also, that the percentage of registration should rise rapidly as the rate of flow increases, so as to be practically correct when the water is drawn from one or more fully open faucets. By this plan, the water-works will be paid on the average for seven-eighths of the total delivery to each consumer, and its business can accordingly be conducted with a reasonable allowance for loss by under-registration, and without unduly increasing the price of the water. A long study of the problem from this standpoint has convinced the writer that the principle just mentioned is sound, and it only remains to be seen whether the indicated degree of sensitiveness can readily be secured by meter makers.

An examination of the various tests submitted and cited by the author, as well as of those previously made by the writer, demonstrates that nearly all the meters mentioned in the paper are capable of meeting the above-named requirement, and hence also that the principal mechanical difficulties in the case have been successfully overcome. This being the fact, there is accordingly no valid reason for manufacturers to complain of unreasonable demands for accuracy by the users of meters, and it only remains for them to devise means of making their products uniform in quality as well as low in price. It should also be stated in this connection that the aforesaid efficiency or "sensitiveness" of 50% registration for so low a discharge as 3 galls. per hour was determined about 9 years ago from numerous experiments with several "Crown" and "Thomson" meters of the small sizes for domestic use, and that the curves obtained by plotting the percentages of accuracy as ordinates to the corresponding discharges in gallons per hour as abscissas were closely alike. Furthermore, these tests showed that in both styles of meter the efficiency rose to considerably more than 90% for a flow of 8 galls. per hour; and as this rate required much less time for making a test than the

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aforesaid lower rate, it was accordingly adopted as a standard for use in the water department, along with a corresponding minimum efficiency of 90 per cent. At discharges of over 50 galls. per hour, all the small meters should register correctly within 1 or 2 per cent.

With respect to durability and maintained accuracy of registration, the results of some recent examinations of small meters by the writer may be of interest. Before being set, all these meters were tested and found to register within the aforesaid limits, and they have remained in place undisturbed for the periods of time indicated. None of them gave evidence of being defective, and they were temporarily removed only for the purpose of ascertaining their condition. The tests for accuracy of registration were all made immediately after removal, and before taking the meters apart for inspecting their interior condition. If found worn or defective in any respect, the necessary repairs were made, whereupon the meters were re-tested and replaced. Unfortunately, no record of the details of the defects and repairs was kept, and hence the results of the examinations are only qualitative. It is also to be regretted that this inspection has hitherto embraced only a few styles of meter, the "Crown" greatly predominating. Furthermore, the water which passed through the meters was in all cases clear and free from any sediment or grit, except such as might come occasionally from a considerable disturbance of the distributing system, such as the repair of broken mains and the addition of new lines of pipe, whereby more or less roiliness of the water is generally produced for short times; also, except the particles of rust or other accretions which form in all cast or wrought-iron pipes and become detached when the flow occurs at an unusually high velocity. Table No. 26, moreover, relates to only a few representative cases of small meters taken at random out of a list of several hundred.

TABLE No. 26.

Style.	Number of examples.	Size.	Number of years in uninterrupted service.	Low-Flow Test.		High-Flow Test.	
				Rate of discharge, in gallons per hour.	Ratio of actual to registered discharge.	Rate of discharge, in gallons per hour.	Ratio of actual to registered discharge.
		Inches.					
Crown.....	4	¾	12 to 16	*41 to 300	1.10 to 1.29	900 to 1 020	1.06 to 1.18
"	3	½	10 " 16	*25 " 150	1.04 " 1.11	900 " 930	1.03 " 1.07
"	4	½	12 " 15	11 " 15	1.09 " 1.14	830 " 930	1.00 " 1.09
"	10	½	10 " 16	10 " 15	0.99 " 1.07	830 " 980	1.00 " 1.06
Hersey....	3	⅝	1 " 3	9 " 13	0.99 " 1.10	980 " 1 300	0.97 " 1.02
Union.....	3	¾	11 " 15	*15 " 50	1.00 " 1.16	1 080 " 1 200	0.98 " 1.00
Thomson...	2	⅞	2 " 4	15	0.98 " 1.19	930 " 1 060	0.97 " 1.00

NOTE.—* Denotes that no registration occurred for lower rates of discharge than those mentioned in the fifth column. In these cases there was always some slight defect of piston or gearing which caused undue friction.

Mr. Kuichling.

An inspection of Table No. 26 shows clearly that the meters mentioned afford little room for adverse criticism, so far as durability and continued accuracy of registration are concerned provided that the water passing through them is practically free from grit, and that proper care is taken in their manufacture. This latter remark is prompted by the fact that in the aforesaid examination a meter was occasionally found which did not exhibit first-class workmanship or materials, and consequently gave a considerably larger ratio of actual to registered discharge, at both low and high flows, than appears in the table. The remedy for this condition necessarily lies with the makers, and can, doubtless, be attained by a more rigid system of shop inspection.

With respect to the design, accuracy and durability of the smaller sizes of meters, it thus appears that the mechanical problem has been practically solved, and that there now remains only the matter of improvement and commercial development. Between the different types or styles there seems to be little choice, except as to cost, original workmanship and facility or economy of making repairs; and in these respects the disc meters possess an undoubted advantage over those which are provided with rotating pistons. The writer, therefore, agrees thoroughly with the author in the belief that most of the water meters now in the market are reliable and durable, and that their use should become an essential feature of all systems of public water supply.

Mr. Rafter.

George W. Rafter, M. Am. Soc. C. E.—The author's proposition, that all water services should be metered, while strongly held to by many water-works managers, has always seemed to the writer to be so far unsettled that it is at any rate still debatable. It is from this point of view that the opposite ground will be briefly traversed in this discussion.

To begin with, the writer agrees that there are always some services in regard to which there can be no question as to the propriety of applying meters. The difficulty appears to be to decide just when to stop. The author says meter everything. To this the writer objects, on the ground that such procedure entails, under the unbusiness-like methods which generally prevail in American cities and towns, a bill of expense, the end of which no one can foresee. The tendency in nearly all municipal business is to employ mediocrity at an ever-increasing scale of wages. The selection of the author, as Water-Works Trustee at Wyoming, is merely the exception that proves the rule, and cannot be taken to indicate any general tendency to substitute business methods in place of political favoritism in municipalities. On this point, however, the writer does not want to be too insistent, because he recognizes that there is now a struggle, in American municipalities, between the spirit of expediency, on the one hand, and the scientific spirit, on the other. In such a struggle the engineer ought to be an

Mr. Rafter. important factor, if for no other reason than that he is the one man in the community with some technical knowledge of municipal business. Indeed, if engineers and other technical people do their part, the struggle between expediency and science may in the end win out in favor of science; and when that day arrives the writer will be more disposed to accept the author's dictum on universal metering.

It seems clear to the writer that, before extensively adopting meters, the cost of maintenance should be carefully considered. Complete statistics, covering all the items of cost for a series of years, are, however, still quite rare, although in a few cities, as at Rochester, N. Y., enough has been learned to indicate the general trend. The meter department in Rochester was established under the direction of the former chief engineer, J. Nelson Tubbs, M. Am. Soc. C. E., who was satisfied that the cost per meter ought not to exceed about $1.50 per year, after the number in use became large enough to give good averages. The system of accounts, as designed by Mr. Tubbs, is very complete, and reference may therefore be made to the Rochester statistics as embodying, probably, as complete information as can be obtained anywhere. That the figures have not justified Mr. Tubb's original view may be ascribed largely to the deterrent effect of municipal politics.

By way of showing the Rochester meter statistics, Table No. 27 has been compiled from the Annual Report of the Executive Board of that city, wherein appear the essential figures of cost for the thirteen years from 1886 to 1897 inclusive.

Column (5) gives the amount received from meters sold to consumers. The Rochester Water-Works adopted the plan of metering some services at public expense; whereas others are metered only at the expense of the consumer, an attempt being made to distinguish between those requiring metering, as a matter of public policy, and those where meters presumably decrease the annual water rate to the consumer, somewhat. In Table No. 27, the cost of these latter is included as a part of the total expense of the meter department, for the reason that, as a matter of final expense to the whole community, it does not matter whether the meters are paid for by the municipality or by individual citizens who are water takers. It seems quite clear to the writer, therefore, that all items of expense attaching to meters so paid for should be included as an element of the final cost of the meter department in any city where the system in vogue at Rochester is pursued. It is fair, however, to separate this item in such statistics as are included in Table No. 27.

As regards the number of services in use at the end of each fiscal year, as per Column (8), it may be remarked that the statistics given in the annual reports since 1890 appear somewhat uncertain, although it is probable that the figures of Column (8) are so nearly right as to

Mr. Rafter.

TABLE No. 27.—Statistics of Meter Department at Rochester, N. Y.

Fiscal Year.		Number of meters in use at end of fiscal year.	Cost of new meters purchased during year.	Amount received from meters sold to consumers.	Total expenditure of Meter Department each year.	Total expenditure of Meter Department, less cost of new meters purchased.	Number of services in use at end of fiscal year.	Ratio of services to meters — Col. (8) ÷ Col. (8).	Approximate population.	Increase in number of meters during year.	Number of new services during year.	Number of meters permanently removed for various causes during year.	Daily use of water per capita. In gallons.			Expenditure per meter.
Day of month ending.	Year.												From Hemlock Lake. Domestic system.	From Holly, Direct pressure, system.	Total of both systems.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
April 5	1886	978	$1 552.45	$662.76	$5 452.85	$3 900.40	15 810	0.062	108 000	87	1 731	.....	55.2	11.2	66.4	$3.99
" 4	1887	1 094	2 582.97	1 144.47	6 468.29	3 885.32	17 454	0.063	113 000	116	1 644	.....	58.5	11.9	70.4	3.55
" 2	1888	1 558	9 648.65	867.38	15 073.23	5 424.58	19 407	0.080	119 000	464	1 953	.....	57.9	12.2	70.1	3.48
" 1	1889	1 700*	5 668.50	1 233.40	11 029.12	5 360.62	21 358	0.073	125 000	146*	1 951	.....	54.3	12.5	66.8	3.15
" 7	1890	2 115	8 912.98	1 322.00	15 517.85	6 604.97	23 000†	0.092	131 000	415	1 642†	.....	51.5	12.9	64.4	3.12
" 6	1891	2 844	14 540.88	5 518.62	24 525.31	9 984.43	24 868†	0.114	136 000	729	1 868	74	49.0	13.3	62.3	3.51
" 4	1892	3 163	4 918.18	3 373.33	16 245.99	11 327.81	25 244†	0.125	140 000	319	376†	131	47.4	12.6	60.0	3.58
" 3	1893	3 460	10 241.07	2 988.00	22 032.38	11 791.31	26 167†	0.132	144 000	297	923†	275	46.1	11.5	57.6	3.40
" 2	1894‡	3 724	21 157.37	3 481.76	33 446.37	12 289.00	26 908†	0.138	148 000	264	741†	218	44.9	12.2	57.1	3.30
" 1	1895	4 043	1 181.16	3 450.89	14 407.16	13 226.00	27 410†	0.147	152 000	319	502†	183	47.4†	11.7	59.1†	3.26
Dec. 31§	1895	5 157	5 245.55	9 6 6.32	16 387.62	11 142.07	28 512†	0.181	156 000	1 114	1 102†	143	50.0†	12.5	62.5†	2.88‖
" 31	1896	6 005	8 729.30	7 564.81	26 021.33	17 292.03	29 500†	0.204	160 000	848	988†	226	60.0†	10.7	70.7†	2.88
" 31	1897	6 497	6 311.30	5 839.14	22 317.42	16 006.12	30 000†	0.216	165 000	492	500†	170	65.3†	8.7	74.0†	2.46
Totals	.....	.....	$100 690.26	$47 132.88	$228 924.92	$128 234.66	.....	.....	.....	.....	.....	1 420	.....	.....	.....	.....
Means	.....	.....	7 745.40	3 625.61	17 609.60	9 864.20	.....	.....	.....	.....	.....	.....	.....	.....	.....	$3.28

* There is a discrepancy of 112 meters in the figures for this year. According to one set of figures in the annual reports, the total number set was 258.

† Approximate.

‡ New conduit from Hemlock Lake in service in October, 1894.

§ A short fiscal year here.

‖ The expenditure per meter for the short fiscal year ending December 31st, 1895 (9 months) was $2.16; $2.88 per meter is on the same basis for twelve months.

Mr. Rafter. permit their use without material error. In the same way, the population figures, as per Column (10), are approximate, except for the census year 1890, in which the total population is given at 133 896. The figures of Column (10) are intended as approximate means for each fiscal year.

The previous remarks in regard to the uncertainty as to the number of services in Column (8) also apply to the number of new services as per Column (12).

The statistics of the number of meters permanently removed for various causes are first given for the fiscal year ending April 6th, 1891. The interesting fact that, in the years from 1891 to 1897, inclusive, a total of 1 420 meters was permanently removed for various causes, is brought out by Column (13), in which these figures appear.

Table No. 27 is so clearly self-explanatory as to render further discussion unnecessary. It seems to the writer to justify clearly the position taken at the beginning of his discussion, that the propriety of universal metering, under existing conditions, in American municipalities is, at any rate, a debatable question. It may be pointed out, however, that the figures of Column (17) do not take into account any allowance for interest on money invested, sinking fund, or for meters permanently removed and going to the scrap heap; nor do they include additional service in the business office on account of the greater complication of meter accounts. Making an addition for these several items, the average cost per meter, for the period covered by Table No. 27, becomes about $4.40, instead of $3.28, as given at the foot of Column (17).

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—The writer is a firm believer in water meters being the best and only equitable controllers of water waste. It is true that some meters will not register small leaks in plumbing, but this fact is not a matter of vital import in the great problem, taken as a whole. The main feature which must not be lost sight of is that the percentage of waste which it is desired to suppress, by the introduction of meters, is not 15 to 20% of the total water supply, but is far more likely to be 65 to 70 per cent. Hence, in a broad sense, it is useless to discuss small percentages. Many writers are continually harping upon the supposed fact that small leaks, due to defective plumbing alone, constitute the main source of water waste. This may be true in isolated cases on a small scale, but, generally speaking, it is far from true, because instances are now too numerous where meters have been suddenly introduced, and, as a result, that infallible test, the coal-bill at the pumping-works has been likewise suddenly reduced as much as 40 per cent. Such close relation between causes and effects leaves little or no room for doubt or discussion.

The main trouble is willful waste. A careful study of the extravagant consumption of water in any modern city, supplying water to

consumers without the use of meters, will generally lead to a sub- Mr. Le Conte.
division of the subject into three categories:

First.—Legitimate consumption equals 40% of the total water supply.

Second.—Willful waste equals 40% of the total supply.

Third.—Defective plumbing and leaky valves and joints in the distribution system equal, say, 20% of the total supply.

Now, the first two can and will be controlled most effectually by meters, while the third will certainly be very materially reduced by the fact that the general adoption of the meter system will itself invariably enforce the introduction of a much higher class of plumbing work throughout the city. This, of course, means a less number of small leaks such as the meters are unable to register.

The accuracy of the meters tested by the author is certainly quite remarkable, and the writer thinks they are really more accurate than the exigencies of the case require. If the meters have an error of 7 to 8%, that certainly cuts no figure in the water-waste problem, and since this error is in all cases, practically, in favor of the consumers, they have no just ground for complaint.

As to the durability of meters, one can only speak in a broad, general way, since local circumstances have great weight in all cases. The average life of a meter, which it is reasonable to expect, depends chiefly upon the character of the water. In localities where the supply is taken and delivered directly from rivers, the waters of which are heavily charged with siliceous sediment, the wear and tear on the meters is, of course, enormous, and the apparatus is practically used up in from 2 to 5 years; the repairs being correspondingly heavy. In all such places it does not pay to furnish meters except to control the supply to the largest consumers. The smaller ones can best be held in check by inspection from time to time. Where the water supply is naturally clean, or is made so by sedimentation; or, better still, by filtration, then the water passing through the meters is practically free from silty materials, and, under favorable circumstances, the life of the meter may and does become very much prolonged, possibly 15 to 16 years or more. In such cases the item of annual repairs will be comparatively light, not to exceed, say, 2 to 3% of the first cost. It may be fairly stated that the grand average useful life of a meter is approximately 10 to 12 years of very fair service, assuming that it is given ordinary attention and care.

A good new meter will register within 1.5 or 2% easily, and the annual increment of this error will generally be something less than 1% —sometimes much less.

The writer agrees with the author in the statement that the only fair and honest way to sell water is by meter measurement.

Mr. Hill. JOHN W. HILL, M. Am. Soc. C. E.—Considering the several criticisms on the paper; some of these undoubtedly are just, and, within the time at the author's disposal, the errors and omissions will be corrected.

When the tests were started they had only one object in view, viz., a comparison, upon a uniform footing, of the less expensive with the more expensive water meters, under all the conditions of daily service liable to occur in the use of meters on domestic service pipes, and, to avoid the chance of error in the work, every precaution was taken, within the limits of the crude apparatus with which the author was compelled to work.

Referring to the temperature of water which was taken for each test: The discharge of the meters being noted in pounds, this weight was divided by the weight of water per cubic foot at observed temperature, to reduce the discharge to the same kind of unit measurement as the registry of the meters, *i. e.*, cubic feet, and it was just as convenient to divide the weight of the contents of the cask by the weight due to the actual temperature of the water as by any other weight. The author does not regard this as a refinement, but as an essential part of the data in a series of tests of this nature.

The scale was in excellent order, and during the experiments was often tested for "sensitiveness" by the addition and subtraction of small weights, and for accuracy, by test weights of known value. The cask was bottle tight, and to prove the absence of leakage was filled, weighed and allowed to remain over night, without change of weight. The apparatus was so arranged that no water could reach the cask unless it came through the meter under test.

The author cannot quite agree with Mr. Smith that the errors of measurement found with the smaller orifices should be excluded in stating the average error of water meters. If the meter would never be required, or infrequently required, to measure the small streams of 10 to 42 galls. per hour, then the average based upon these and larger streams together, would be manifestly fair; but it is a fact generally recognized that the leaks in domestic water appliances are conditions that must be met if the water meter does its whole duty; and it is the supposed incapacity of water meters to properly account for the small streams represented by leaks which has been one of the objections urged by water-works managers to the general adoption of the meter system on domestic service pipes.

The author is heartily in accord with Mr. Kuichling's view, that a reliable water meter should give a reasonable account of the leaks of pipes and fittings in domestic plumbing, and it is gratifying to know that the modern meter is abundantly capable of doing this, with a degree of accuracy which, in some instances (shown by Table No. 1), is really marvelous.

The four Trident meters tested were not provided with extension dials, and only one of these (No. 23 176) had been in service prior to these tests, but this meter was taken off the service pipe and tested without opening and cleaning it, and, except for the small stream of 11 galls. per hour, gave quite as good results as the unused meters. The simplicity of construction, strength, and general adaptation to practical uses of this meter appealed so strongly to the author as to justify him in recommending it as one of the few meters to be adopted for use by the Trustees of the Wyoming Water-Works. Mr. Hill.

With reference to the test of strength of the working parts of meters by suddenly applying water pressures of 100 lbs., more or less, it must not be overlooked that this was an unbalanced pressure, the pressure on the outlet side of the meter at such times being the atmosphere. With a meter connected in the line of service pipe such a test as Mr. Thomson very properly suggests could have no significance, because the pressure upon both sides of the meter would be approximately alike, but the sudden application of this pressure to a meter with no resistance above the atmosphere upon the outlet side, subjects it to a shock which will seldom occur in practice, and if the meter is not well built, such treatment, often repeated, is very liable to injure it.

Mr. Thomson has misunderstood the author with regard to the character of the water passed through these tested meters. The Wyoming water is of high purity, and carries no sand or grit, and excepting the salts of iron, it contains no material calculated to have any deleterious influence on the long-continued operation of a water meter. The 3-in. hot-water Worthington meters, however, were tested with Ohio River water, which, among other inferior qualities that it possesses, carries a large amount of gritty suspended matter. Mr. Thomson also regards the obstruction of meter or loss of head generally as of no great importance, while Mr. Smith thinks the importance of the loss of head cannot be overestimated. This wide disagreement of two eminent experts on water meters is very painful to the author, and he would suggest that if these gentlemen will get together and reach a compromise view, he will gladly accept their joint decision. Meanwhile, the author is of the opinion that, without injury to the meter itself, efforts should be made to secure a construction which will absorb the least portion of the static head in overcoming meter resistances.

It was thought by the author that the figures in the column headed "Ratio of Tank to Meter" in Table No. 1, would readily convey to the reader's mind a correct conception of the relation of the meter registry to the actual quantity of water discharged, without the aid of diagrams. To remedy this omission, the curves on the diagrams, Figs. 17 and 18, have been plotted with the reciprocals of the quanti-

Mr. Hill. ties in the seventh column of the table, and the discharges of the meters in gallons per minute, as co-ordinates, Fig. 17 containing the data obtained from the tests for sensitiveness of the piston and inferential meters, and Fig. 18 the same data obtained from the disc meters.

The author's reason for comparing the actual discharge of a meter for any given standard orifice, with the discharge obtained from the curve, is as follows:

During the tests for accuracy of measurement and sensitiveness to varying rates of discharge, the pressures were taken only on the inlet side of the meters. From these tests the data of Table No. 1 were obtained. Afterward the plain pipe was substituted for the meters and the data of Table No. 6 obtained; and finally the apparatus was further altered to take pressures on both sides of the meter, and the data of Table No. 5 were noted, but at this time, owing to press of other duties, the notes were limited to the pressures on the inlet and outlet pipes, and no note was taken of the reading of the meter or the actual discharge of water.

The author, however, had no reason to believe that any change occurred in any meter which would cause its discharge for a given pressure on the inlet side and given standard orifice, to vary from the discharge for the same pressure and orifice when the notes for Table No. 1 were being taken, and, for example, assumed that with 29 lbs. pressure on the inlet side of the Worthington meter, the discharge was the same as when this pressure occurred during the tests for accuracy and sensitiveness.

The curves were based upon the actual discharge at the pressures noted in Table No. 6, and are theoretical only so far as the well-known laws of hydraulics are held to apply to the discharges of the plain pipe by pressures at the inlet side, above and below the pressures noted in the table.

Upon reflection, however, the author thinks Mr. Thomson may be correct in assuming that the discharge of the meters should be compared with the discharge of the plain pipe, and not with an assumed discharge of the pipe under the pressure which was had on the inlet side of the meters during the tests, for such pressure could not obtain with the plain pipe, because more of the static head is expended in producing velocity, and less in overcoming frictional and other resistances. Therefore, the assumed head on the plain pipe is an impossible head under the conditions of the author's tests, and could be had only by a large increase of the static pressure on the service pipe, which in a reservoir water-works is not conveniently to be had. With this explanation, the author submits Table No. 28, showing the relation of the actual discharge of the meters to the possible discharge of the unmetered service pipe. This table ignores the differences of pressure

Mr. Hill.

Accuracy Test of $\frac{5}{8}$ inch Meters.

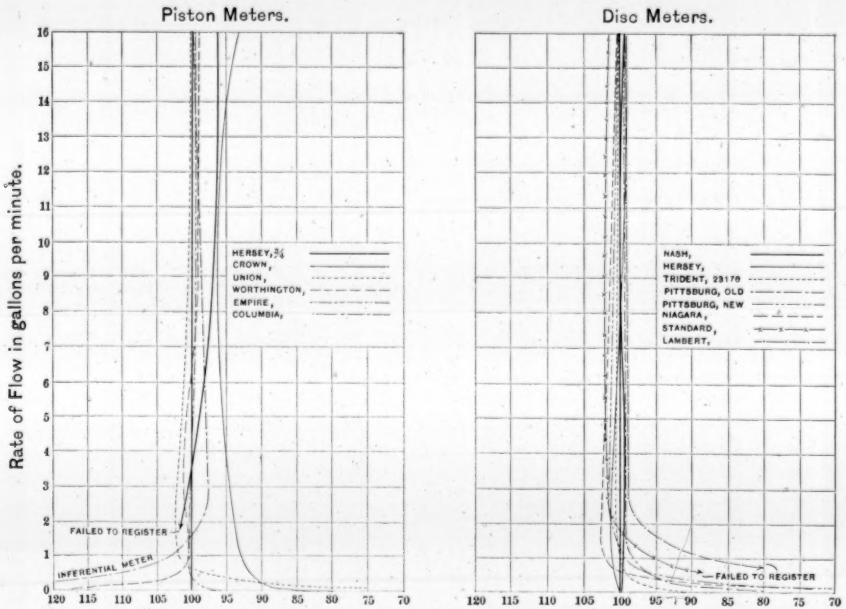

FIG. 17. FIG. 18.

Mr. Hill. on the inlet side of the meter and on the inlet side of the plain pipe. The data for Table No. 28 are derived from Tables Nos. 6 and 7.

TABLE No. 28.—REDUCED DISCHARGE OF SERVICE PIPE BY INTRODUCTION OF METERS.

Meter.	Gallons per hour, by meter.	Gallons per hour, plain pipe.	Percentage of reduced capacity.
PISTON METERS.			
Worthington	828.27	.....	86.14
Crown	872.85	.....	90.78
Hersey	913.52	961.49	95.01
Empire	746.60	.....	77.65
Union	940.94	.....	97.86
Worthington	531.30	.....	92.02
Crown	546.31	.....	94.62
Hersey	559.95	577.37	96.98
Empire	503.90	.....	87.28
Union	581.90	.....	100.78
Worthington	172.25	.....	98.76
Crown	172.68	.....	99.00
Hersey	172.58	174.42	98.95
Empire	167.44	.....	96.00
Union	178.06	.....	102.09
DISC METERS.			
Nash	933.14	.....	97.05
Hersey	868.87	.....	90.37
Trident, No. 23 178	926.78	.....	96.39
Trident, No. 23 179	900.72	.....	93.68
Trident, No. 23 180	873.18	.....	90.82
Trident, No. 23 176	915.81	961.49	95.25
Pittsburg, No. 8 011	896.92	.....	93.28
Pittsburg, No. 10 798	907.36	.....	94.37
Niagara	894.93	.....	93.08
Standard	938.18	.....	97.58
Lambert	911.27	.....	94.78
Nash	573.79	.....	99.38
Hersey	556.68	.....	96.42
Trident, No. 23 176	556.23	.....	96.34
Pittsburg, No. 8 011	548.74	577.37	95.04
Pittsburg, No. 10 798	557.06	.....	96.48
Niagara	548.71	.....	95.04
Standard	571.70	.....	99.02
Lambert	560.57	.....	97.09
Nash	172.70	.....	99.01
Hersey	172.11	.....	98.68
Trident, No. 23 178	173.52	.....	99.48
Trident, No. 23 179	169.19	.....	97.00
Trident, No. 23 180	167.95	.....	96.29
Trident, No. 23 176	174.50	174.42	100.05
Pittsburg, No. 8 011	175.71	.....	100.74
Pittsburg, No. 10 798	170.71	.....	97.87
Niagara	168.95	.....	96.86
Standard	175.23	.....	100.46
Lambert	173.03	.....	99.20
INFERENTIAL METER.			
Columbia	762.43	961.49	79.30
Columbia	511.99	577.37	88.68
Columbia	172.93	174.42	99.15

The anomaly is presented in this table of some of the meters passing more water than the unmetered pipe, but this, it will be observed, upon referring to Tables Nos. 6 and 7, is due to a greater static head

Mr. Hill.

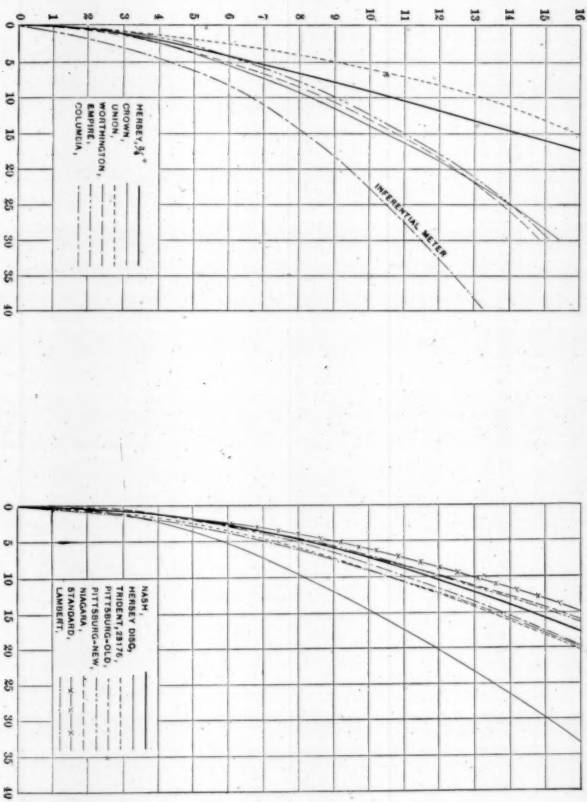

FIG. 19. FIG. 20.

Mr. Hill. on the service pipe at times of testing certain meters than at the time of testing the discharge of the plain pipe.

The percentage losses of head in Table No. 5 and the percentage reduction of capacity of the service pipe by meter (Table No. 7) were intended only to show what these data were under the conditions of the tests described by the author. To remedy one objection to Table No. 5, the author has computed the discharge of the meter in gallons per minute, which, with the losses of head in pounds per square inch, given in column 5, furnishes the co-ordinates of the curves on the diagrams (Figs. 19 and 20).

It is true that the introduction of the meter system for water measurement represents to any community a comparatively large outlay, and the maintenance of the system a continual expense, but the author believes that the equity in charges for water service by the meter system will be justified, both to the consumer and the municipal corporation, and in the end both will be gainers. The practice of allowing "John Smith" to draw and waste all the water he can from a given service pipe, by paying so many dollars per year, answered the purpose in the days of high water charges and expensive water meters, but with a constant shrinking of charges for water, and low-priced, accurate, durable water meters, this primitive practice should no longer prevail, and measurement of the water actually drawn should be substituted for a wild guess as to the probable consumption by an average household.

The statistics, from the records of the Rochester Water Department, of water consumption and meter measurement, presented by Mr. Rafter, are very interesting and instructive, and might be amended by another column in the Table (No. 27) showing the saving to the water department in those instances where meters are used, by the substitution of meters for survey rates. Aside from the fact that the universal metering of service pipes gives the water-works manager a complete command of the commodity which he is distributing, and proportions the burden of cost among the customers more equitably than by the survey system, it admits of such an adjustment of the water rates as will produce the revenue necessary for the proper support of this branch of the municipal government, without favoring one class of consumers at the expense of another.

Referring to the consumption of water per average household. This is a matter which admits of wide discussion. Data have come into the author's possession which show the daily per capita consumption to range from 7.5 to 22 galls., including all the usual purposes to which water is applied about a modern residence. These figures correspond to 2 200 and 6 440 cu. ft. per year for a household of six persons. Mr. Smith estimates 10 000 cu. ft. as the average annual work of a $\frac{5}{8}$-in. water meter, while Mr. Harlow finds from five years' records for Wil-

kinsburg, Pa., an average consumption of 7 500 cu. ft. per annum per household. Mr. Kuichling's estimate of 60 galls. per capita per diem represents an average consumption of 17 570 cu. ft. per annum for a household of six persons, but this, doubtless, is intended to include the public consumption of water for street sprinkling, sewer flushing, extinguishment of fires, and perhaps other uses in addition to the purely domestic consumption by a family. Dexter Brackett, M. Am. Soc. C. E.,* from investigations in Boston and other cities, furnishes data for the following annual minimum and maximum consumption for a family of six persons: Mr. Hill.

	Cubic feet per annum.
Boston, Mass	4 860 — 17 274
Worcester, Maas	3 572 — 6 850
Fall River, Mass	2 460 — 7 466
Newton, Mass	2 020 — 7 759
Yonkers, N. Y. (average)	6 266

The author's experience indicates that the consumption for purely domestic purposes by a family of six persons will vary from 4 500 to 10 700 cu. ft. per year. The larger consumption, in all instances, is for houses fitted with modern plumbing, and the use of water for private stables, and the smaller consumption for houses with less pretentious water appliances and the consumption of water limited to house use.

In closing this discussion the author wishes to thank the members of the Society who have so earnestly and ably partieipated in the discussion of a subject of constantly growing interest and importance to municipal corporations and water consumers. He felt that there must be locked up somewhere a large amount of valuable information upon the "accuracy and durability in service of water meters" and that a random shot, well sped, might discover some of it. That this opinion was well founded is demonstrated by the valuable matter brought out in the discussion of this brief paper. Doubtless other papers and discussions of this topic in the future will go far toward overcoming the hesitation of many water-works officials in the general adoption of the meter system of water measurement.

* *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 189.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 849.

SOME EXPERIMENTS ON BRIDGES UNDER MOVING TRAIN-LOADS.

By F. E. TURNEAURE, Assoc. Am. Soc. C. E.

PRESENTED DECEMBER 7TH, 1898.

WITH DISCUSSION.

INTRODUCTORY.

All specifications for railroad bridges recognize in some way the principle that more liberal provision should be made for the stresses due to moving loads than for those due to fixed loads. In some specifications the method of treatment appears to be based entirely upon the theory of the fatigue of metals, and the intensity of stress is determined by a formula of the form: $p = a\left(1 + \frac{\text{Min.}}{\text{Max.}}\right)$. In others the element of fatigue is not considered, but the live-load stresses are increased by a certain percentage "for impact," and then treated as dead-load stresses, thus assuming that the only difference between the effects of live and dead loads arises from the fact that the actual stresses produced by moving loads are greater than those produced by the same loads when stationary. Still other specifications take account of both fatigue and impact.

While all engineers are agreed that some extra allowance must be made for live loads, yet there exists the widest difference of opinion as to how much this allowance should be and how it should be distributed among the different members of the bridge. It is certainly logical that if both fatigue and impact exist, both should be provided for, although, perhaps, by a single formula; but before such a formula can be determined the two effects must be considered separately. Considerable agreement has been reached in the matter of fatigue formulas, but, owing to the lack of reliable data, impact formulas are, at the present time, largely a matter of guess work.

In the hope of contributing something to the solution of this problem, the author, during the summer of 1897, carried out a series of experiments on several bridges of widely varying span and design; and it is the purpose of this paper to present the results of these tests, together with a description of the apparatus used, and a discussion of the points involved.

APPARATUS.

The instruments used were three in number and of two kinds, viz., one instrument for measuring the deflection of the structure as a whole, and two instruments for measuring extensions and compressions of individual members.

The Deflectometer.—The instrument used for measuring deflections was a Fraenkel deflectometer,* the design of the late Professor Fraenkel of Dresden, Germany. Plate IX is a photograph of this instrument showing it attached as it would be in actual use on a plate girder. In this type of instrument the main part of the apparatus is fastened to the bridge, and connection is made by means of a wire, to a heavy weight placed on the ground directly underneath. This wire is made fast, through an interposed steel ribbon, to the circumference of a thick disk, or wheel, mounted on the framework, which disk is cast in one part with another of twice its diameter and having the same axis. On the larger disk is fastened a second steel ribbon attached at its other end to the circumference of a spirally shaped, hollow wheel containing a clock spring By means of a crank the spring may be wound up so as to produce in the steel ribbon a tension sufficient to hold the vertical wire taut, while at the same time allowing free movement of the wire

*Cuts of a slightly different form of this instrument are shown in *Der Civilingenieur*, 1884, Plate XXX; also in *Engineering News*, February 8th, 1894, p. 115.

relative to the frame of the instrument. The recording pencil is attached to the short steel ribbon previously mentioned, and it has, therefore, a motion relative to the frame of the apparatus twice as great as the motion of the bridge. A spindle is provided for holding a roll of paper; another one upon which the paper is wound, and a drum over which the paper passes, and along which the pencil moves as the bridge moves up and down. The paper is moved by electrically controlled clockwork placed inside the drum. The governor of the clockwork was so regulated as to make the rate of motion of the paper about $\frac{4}{10}$ in. per second. The apparatus is very compact, and can be fastened by means of heavy, pointed screws to any horizontally projecting plate, or to a flange of a bridge post. A second pencil was put on this instrument for the purpose of recording electrically the passage of the wheels of the trains, but the operating magnets did not work satisfactorily, and the desired information was obtained mainly from similar pencils attached to the other instruments.*

The Extensometers.—The instruments used for measuring strains in the members were also of a form invented by Professor Fraenkel, but greatly improved by Oscar Leuner, of Dresden, the maker of all the Fraenkel instruments.†

Plate X is a photograph of the recording part of one of the instruments used. In this form of extensometer the longitudinal distortion of a certain length of a member is multiplied about 140 times by means of a compound lever, the motion being communicated to a pencil which traces a line on a moving strip of paper, as in the deflectometer. Although this instrument was designed some seventeen years ago, yet it is only within the last two or three years that it has been so constructed as to reduce the friction of the rapidly moving parts to such an extent as to make it sufficiently sensitive for good work. As now constructed,

* Another deflectometer of this type is that invented and used by M. Rabut. It is illustrated in *Annales des Ponts et Chaussées* of 1896, Vol. xii., p. 381, in a very complete article, in which this and other instruments are described, and their use in the testing of bridges discussed. In this instrument the wire is held by a long coiled spring attached to a separate point of the bridge, and the relative motion is imparted to the recording pencil by attaching the wire to the short end of a lever, the pencil being at the other end. The lever is made very light, and, on the whole, the arrangement is probably better, although not so convenient, as that of the Fraenkel apparatus. By an ingenious arrangement of two weights, and two wires which intersect near the recording apparatus, M. Rabut is enabled to record lateral as well as vertical movements.

For descriptions of examples of the other type of deflectometer—that in which the recording parts are supported from the ground and the pencils are attached to the structure—see "Vibration of Bridges," by S. W. Robinson, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xvi, 1887, p. 42; and "Bridge Deflections," by M. A. Howe, M. Am. Soc. C. E., *Journal of the Association of Engineering Societies*, Vol. xiv, 1895, p. 513.

† For a description and illustration of the original form of instrument, see *Der Civilingenieur*, 1881, p. 249.

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
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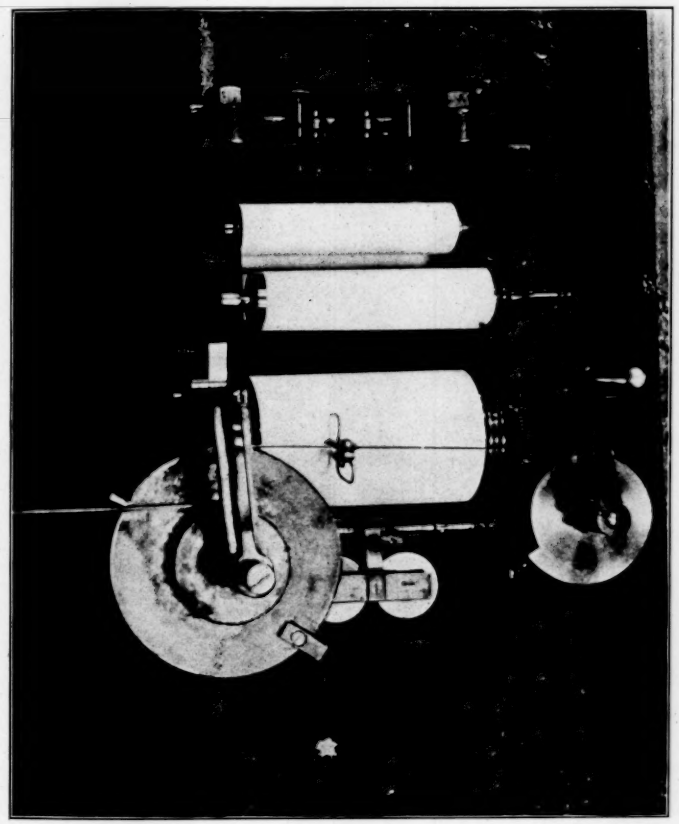
TURNEAURE ON MOVING TRAIN-LOAD EXPERIMENTS.



all bearings are knife-edge, and all connections are made by means of steel ribbons attached to cylindrical surfaces. The levers are mounted in a suitable framework, which carries also the clockwork mechanism for moving the paper, and which is provided with screws for attaching the apparatus to the bridge member. The recording pencil is held by a light carriage running parallel to the axis of the drum on which the paper is stretched. This carriage is moved by two small steel ribbons attached to a sector at the end of the lever system, which arrangement gives rectilineal motion to the pencil. To the short end of the lever system is fastened one end of a long, light rod, clamped at its other end to the bridge member. In the instruments as used, these rods were of such a length that, when properly adjusted, the apparatus would measure the distortion of one meter's length of the bridge member. The multiplication of the distortion of the bridge member was such that, assuming the modulus of elasticity to be 29 000 000 lbs. per square inch, 1 in. on the diagram corresponds to a stress of 5 600 lbs. per square inch, which value has been used in the computations. Additional pencils for recording the passage of wheels were also attached to these instruments, and in this case they worked satisfactorily.

In order to make the friction of the pencil on the paper as small as possible, metallic paper and aniline pencils were used, these being furnished with the instruments. With this arrangement a distinct mark is made with an exceedingly light pressure, and the pencil does not wear smooth as lead does. The speed of the paper in the extensometers was, in one case, about $\frac{3}{10}$, and in the other case, about $\frac{5}{10}$, in. per second. The speed was determined at frequent intervals, and was fairly constant; but, owing to the method of governing the clockwork, it could not easily be regulated to any particular value. The speeds of trains were determined from the records of the additional pencils above mentioned, the speed of the paper and the length of the locomotive wheel-base being known.*

*Another extensometer which has been used to a considerable extent in France is that known as the Manet apparatus. It has been perfected by M. Rabut, and used quite extensively by him. The instrument is not a self-recording one, the motion being multiplied and communicated to a pointer moving over a dial. It is illustrated and described in the article by M. Rabut, before mentioned; also, in Johnson's "Framed Structures," p. 230.

Another instrument which should be mentioned in this connection is that constructed by Messrs. J. J. Hankenson and Wm. H. Ledger, and used by them in carrying out a series of tests at Cornell University. In this instrument a much less multiplication is used than in either of the others. Single levers are used, and a continuous record obtained. *Engineering News*, Vol. xxxiii, 1895, p. 300.

Track Instrument.—The electric track switch used was a rather crude affair, but answered the purpose very well. Some difficulty was experienced in devising an apparatus which would work slowly enough under fast trains. That which was finally successful was made as follows: On a long wooden block a saw-blade was placed and loosely held at its two ends under thin metal straps. This blade was then sprung upward and supported on brass springs along the center, and so arranged that on being depressed about ½ in. an electric circuit would be closed. This contrivance was fastened just outside the rail and parallel thereto, and at such an elevation that a passing wheel would keep the circuit closed while moving a distance of 12 or 15 ins.

General Description of the Experiments.

The experiments in question consisted in obtaining records of stress and deflection under moving trains from a number of bridges on the Chicago, Milwaukee and St. Paul, and the Union Pacific railroads. Tests were made on twelve plate-girder bridges, of spans varying from 25 to 80 ft. in length, and on eleven truss bridges of spans from 100 to 200 ft. in length.

The chief characteristics of these bridges are given in Tables Nos. 1 and 2. All the plate girders are through bridges, and, with the exception of Nos. 10 and 12, the ties are supported on shelf angles. They are, no doubt, somewhat shallower than deck girders of like span, and, therefore, somewhat more flexible. All the girders, however, are of recent construction and represent modern practice. All the truss bridges are of the Pratt type, except Nos. 20 and 21, which are Pegram trusses. The dates of erection indicate fairly well the character of the construction, Nos. 13 and 15 being light structures; while Nos. 14, 17, 18 and 19, are fairly heavy, modern, Pratt trusses. The two Pegram trusses have long panels and a suspended floor system with plate hangers. The last four bridges are on the Union Pacific, all others are on the Chicago, Milwaukee and St. Paul Railway.

In the testing of girder bridges, the deflectometer was usually fastened to a flange at the center of the bridge, and one or both of the extensometers attached to the flange of the girder as near the center as possible. For the sake of comparison, simultaneous measurements of stress were frequently made, in some cases on the two lower flanges, and in others on the outside lower and outside upper flanges. In gen-

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
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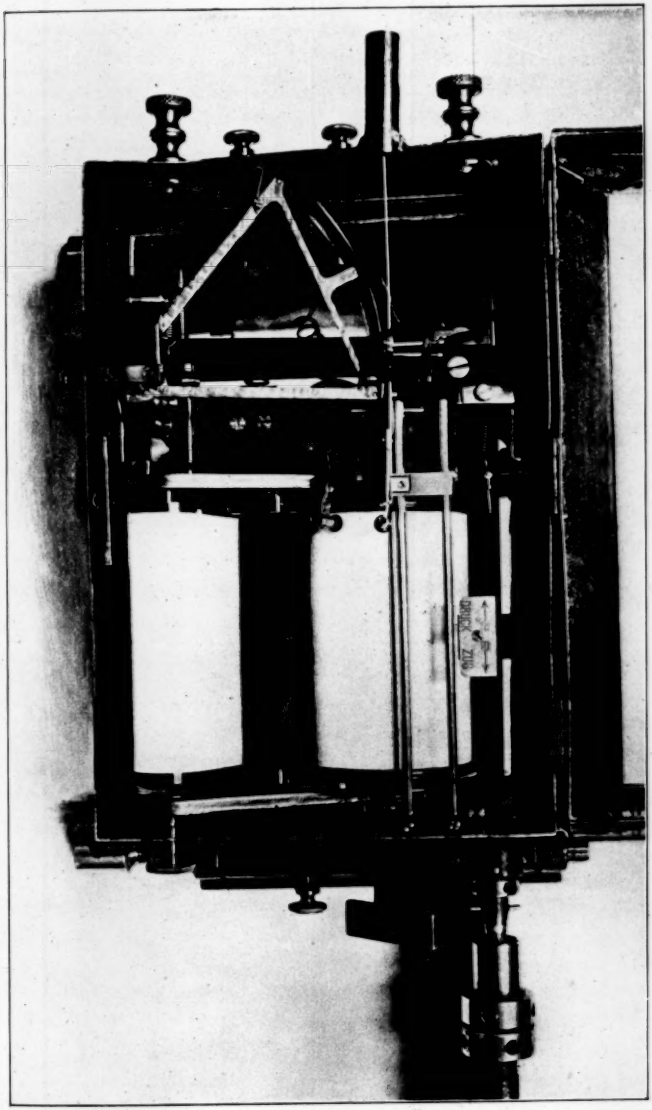
TURNEAURE ON MOVING TRAIN-LOAD EXPERIMENTS.



eral, about twelve or fifteen experiments were made on each girder bridge, this being usually a sufficient number to bring out as great a variety of speeds and locomotive effects as would be likely to occur with any frequency. In most of the girders the effect of the locomotives was soon seen to be the feature of chief interest, that of either passenger or freight cars not being of special importance.

In the experiments on trusses the deflectometer was usually attached to the post nearest the center of the bridge, and records were obtained from it in that position for all tests on the structure. At the same time the extensometers were attached to various members of the bridge. After a little experience it was concluded that the members from which the most significant curves could be obtained were: A lower chord member, the main diagonal nearest the center, and the hip vertical; and these were, therefore, the members usually experimented upon. By the use of both extensometers determinations were made, in many cases, of the bending in individual bars and of the relative tension in the various bars of the same member. Such experiments, however, were made of secondary importance, the main object of the work being to secure a reasonable number of diagrams from as many different structures as possible. As a rule, from thirty to forty experiments were made on each truss.

In all, about four hundred experiments were made, three curves usually being obtained from each. With few exceptions, the experiments were made with the trains of the regular traffic. As in most cases there were but ten or twelve movements during the day, not as many diagrams were obtained as might have been desirable; but it was thought that the work would be more valuable if a number of structures of different span-lengths were tested than if a great number of experiments were made on two or three bridges. This idea was strengthened by the fact that in those places where a longer stay was made at a structure, very little which was new was observed in the deflection curves after the first day's work.

On two different occasions, through the kindness of the railroad officials, a short freight train was placed at the disposal of the experimenters. In one case the train consisted of an American locomotive, one lightly loaded car, and three empties; and with this, Tests Nos. 97 to 107 were made on Bridge No. 14. In the other case it consisted of a ten-wheel engine, four heavily loaded cars, and a caboose; and with this

train tests were made on Bridges Nos. 11, 12, 18 and 19. The use of a special train in making such tests is obviously of great advantage, both in enabling the tests to be made very rapidly, and in facilitating comparisons. For the tests with special trains, and for many of the others, stress curves were computed, the data regarding weights of locomotives and cars being furnished by the railroad companies. In the computations the influence-line method was used as being the only practicable one for such a large amount of work. The results of the experiments are presented and discussed further on.

Working of the Apparatus.

The working of the apparatus was in general quite satisfactory. The deflectometer proved very convenient to adjust, and was very reliable in its working, except for the short-girder spans. With spans less than about 50 ft. in length, the jarring of the girder agreed so closely in period with that of the moving parts of the instrument (about 20 per second) that the apparatus was set into excessive vibration, and the resulting records in such cases are not of much value. Some reliable curves, however, were obtained from a 25-ft. span by placing the instrument on the ground and making the connection by means of gas pipe. These curves show that the vibration or jarring of the bridge at this rapid rate was really of very small amplitude, a very small fraction of the disturbances of a longer period which are discussed further on. In the longer girders and in the truss bridges this jarring action nearly disappeared, and the curves obtained from the deflectometer bring out very clearly the vibration of the structures without interference by instrumental vibrations. If the moving parts of the instrument were made much lighter, it is believed that it would be much more useful for testing very short spans.

The moving parts of the extensometers, owing to the high degree of multiplication of the instruments, constitute a rather flexible system, and have, therefore, a rather slow natural rate of vibration, much slower than the deflectometer. This rate of vibration was found experimentally to be about 10 per second, and, therefore, where the variation in stress about equals or exceeds this in rapidity, the records of the instruments are not reliable. Since the rates of vibration of the strucures, or of individual members, were in most cases much less than this, little trouble was experienced on this score. The jarring action

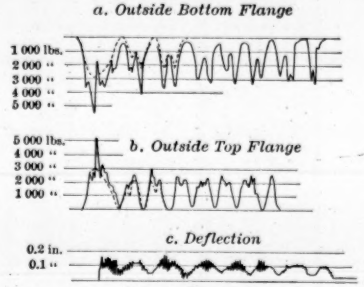


FIG. 1

TEST NO. 161., BRIDGE NO. 3.

PASSENGER TRAIN WEST; 47 MILES PER HOUR.

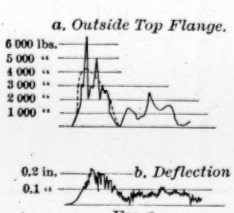


FIG. 2

TEST NO. 55., BRIDGE NO. 5.

FREIGHT TRAIN EAST; 39 MILES PER HOUR.

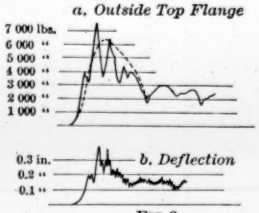


FIG. 3

TEST NO. 72., BRIDGE NO. 6.

FREIGHT TRAIN EAST; 27 MILES PER HOUR.

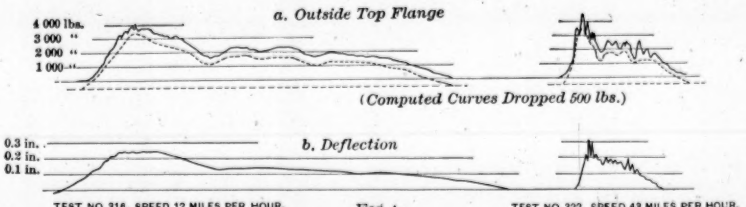


FIG. 4

TWO TESTS ON BRIDGE NO. 11, WITH SPECIAL FREIGHT TRAIN GOING EAST

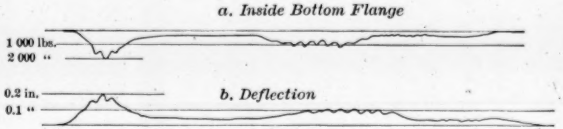


FIG. 5

TEST NO. 86., BRIDGE NO. 10.

FREIGHT TRAIN WEST; 40 MILES PER HOUR.

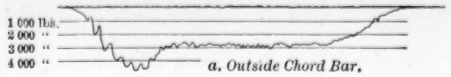
which interfered with the working of the deflectometer appeared to be much too rapid or too small to affect the extensometers, as these instruments invariably gave curves free from such vibrations. The fact that the rates of vibration of the two kinds of instruments were so much different furnished a valuable check to the workings of both.

In order to make the rate of vibration of the extensometers as rapid as possible, the steel carriages supporting the pencils were replaced by much simpler ones made of aluminum. It would, however, be desirable to reduce the weights of the rapidly moving parts still further. The author does not believe it possible with any form of apparatus yet devised to record correctly all the changes in stress which occur in certain members; as, for example, flanges of short girders and stringers, and possibly main members of trusses. From observations on deflection it is believed that these rapid changes, accompanied by a jarring sensation, are very small; but they can only be measured by an instrument whose moving parts are almost without weight. A reference to some of the curves obtained in the experiments of Messrs. Hankenson and Ledger, referred to further on, will show several places where this same effect of instrumental vibration has, doubtless, entirely masked the real distortion in the members.

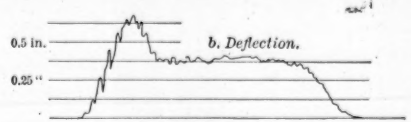
General Characteristics of the Curves.

On pages 417, 419 and 421, are given several curves showing the main characteristics of the principal diagrams taken. Ordinates on the deflection curves indicate deflections in inches; ordinates on the stress curves indicate stress per square inch. The dotted lines are computed curves. The diagrams shown on page 417 are from plate girders, while those on pages 419 and 421 are from trusses. Figs. 1, 2 and 3, were taken from short girders of 30, 35 and 45-ft. span-lengths, respectively; Fig. 4 from a 52-ft. span, and Fig. 5 from a 70-ft. through girder. The details of the various bridges are given in Table No. 1.

In general, the curves from plate girders taken from rapidly moving trains show, in the part of the diagram caused by the locomotive, large projections, or vibrations, usually corresponding in period to the revolution of the drivers, and due no doubt to the action of the counterweights (see Figs. 1, 2 and 3). On the shorter spans this action was so sudden and severe that the pencils of the extensometers were often evidently thrown too far, as can be seen by comparison with

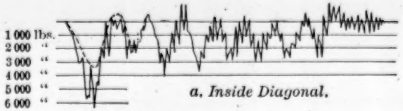


a. Outside Chord Bar.

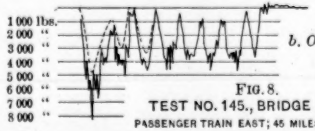


b. Deflection.

FIG. 6.
TEST NO. 13., BRIDGE NO. 13.
PASSENGER TRAIN WEST; 40 MILES PER HOUR.

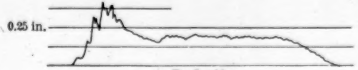


a. Inside Diagonal.

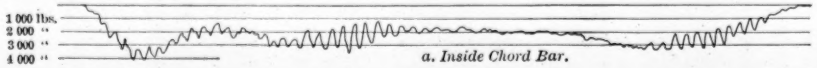


b. Outside Hip Vertical.

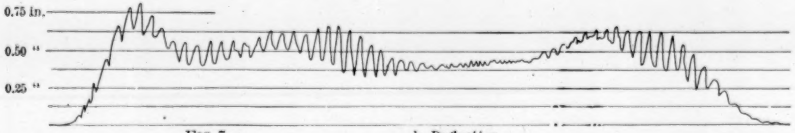
FIG. 8.
TEST NO. 145., BRIDGE NO. 15.
PASSENGER TRAIN EAST; 45 MILES PER HOUR.



c. Deflection.



a. Inside Chord Bar.



b. Deflection.

FIG. 7.
TEST NO. 14., BRIDGE NO. 13.
FREIGHT TRAIN EAST; 28 MILES PER HOUR.

the deflections (Fig. 2). For the longer spans, as in Fig. 5, this effect of drivers diminished very much and the curves from the two instruments agreed better. Where the speeds were less than 15 or 20 miles per hour, the vibrations were small. In Figs. 1, 2 and 3, the deflection curves show the effect of the jarring action before mentioned, and in Fig. 1 the vibrations are undoubtedly much exaggerated by the instrument. As a rule, the larger the bridge, the less this effect, and in the case of the two girder spans of the through type it did not appear. Passenger cars produced no noticeable vibrations in girders other than this jarring action, and the same is true of freight cars on all the girders except the two through spans (see Fig. 5).

Specimens of the diagrams obtained from truss spans are shown in Figs. 6 to 11 inclusive. In very many cases considerable vibration of the truss was caused by the locomotive, as shown by the deflection diagrams in nearly all the figures. This was true for passenger as well as for freight locomotives, and for a considerable range of speed, although not usually caused by trains running slower than 20 miles per hour. Passenger cars never caused large vibrations, nor did empty freight cars, but loaded freight cars often did, as in Fig. 7. Diagrams of chord stress are seen to be quite similar to the corresponding deflection curves. Those for stress in the diagonal and in the hip vertical show, of course, large variations due to the wheel-load concentrations, and, besides, exhibit very large vibrations under the locomotive, which correspond usually to the vibrations in the deflection curves. The rapid vibrations shown in the stress curves for the diagonals in Figs. 8, 9 and 10 are due to the vibration of the individual bars. These are much more rapid than those caused by the vibration of the bridge as a whole. The agreement between the computed and the observed stresses varies; but in general the curves from the instruments are similar to the computed curves, although the absolute values of the ordinates differ quite materially on account of secondary stresses.

Discussion of the Experiments.

From an examination of the curves and a consideration of the elements affecting the problem, it was thought that the discussion could best be divided into the following parts: Effects of Speed Alone; Vibrations in Girders and Trusses; Relation of Stresses to Deflections, Secondary Stresses and Computed Deflections; Results of Other Experiments; Conclusions.

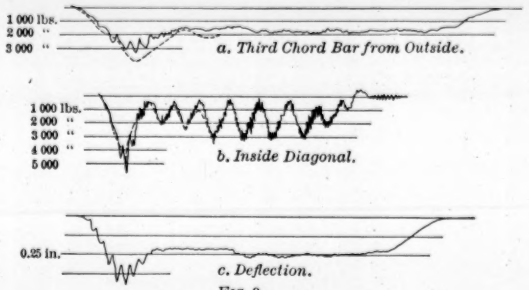


FIG. 9.
TEST NO. 185., BRIDGE NO. 16.
PASSENGER TRAIN EAST; 37 MILES PER HOUR.

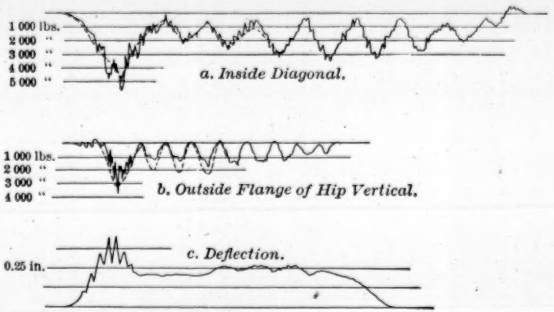


FIG. 10.
TEST NO. 201., BRIDGE NO. 16.
PASSENGER TRAIN EAST; 31 MILES PER HOUR.

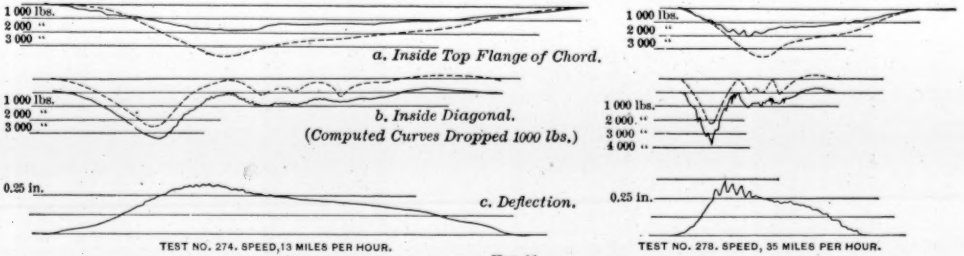


FIG. 11.
TWO TESTS ON BRIDGE NO. 19, WITH SPECIAL FREIGHT TRAIN GOING EAST.

Effects of Speed Alone.

Inasmuch as in discussions on impact, greater stress is usually laid upon the feature of the rapidity of application of the load than upon vibrations, and, as impact formulas appear to give emphasis in this same direction, it was thought desirable to discover, if possible, the effects of speed alone, when separated from those of vibration.

Experimental Results.—If a train moves over a bridge on a perfectly smooth track at any given speed, and if all the wheels are truly circular and perfectly balanced, it is evident that there will be produced a smooth deflection curve, which, when compared with the curve made by the same train when crossing at a very slow speed, will bring out clearly the effects of the rapidity of application of the load. In the actual case various elements conspire to cause vibrations, which are, if of any considerable duration, of equal amplitude each side of the imaginary smooth curve previously mentioned. From these considerations it was concluded that curves constructed by drawing middle lines through the sinuous lines of the diagrams, could be fairly compared in studying the effects of speed alone.

In making these comparisons, two methods were used. One was to measure the mean ordinate to the curve of the extensometer at some particular point and then compare with the computed stress. The other was to compare the curves from both the extensometer and deflectometer caused by the same train at different rates of speed. The latter is by far the more reliable and satisfactory, as it eliminates many uncertainties in the matter of weights and dimensions, but it requires the use of a special train for testing, which was had only for the few bridges already mentioned.

The results of this comparison for eight bridges are given in Figs. 12, 13, and 14. In all cases the stress in the flange, or other member, was computed for that position of the locomotive which would give the maximum stress. Mean ordinates were then measured to the corresponding points on the diagrams, and these measured stresses then compared with the computed. The ratios of the observed to the computed stresses are plotted in the figures as points, ordinates representing the values of the ratios, and abscissas representing velocities in miles per hour. Dotted horizontal lines representing average ratios have been drawn in those cases where the points could not readily be connected. It was not to be expected that the observed stresses would agree very closely with

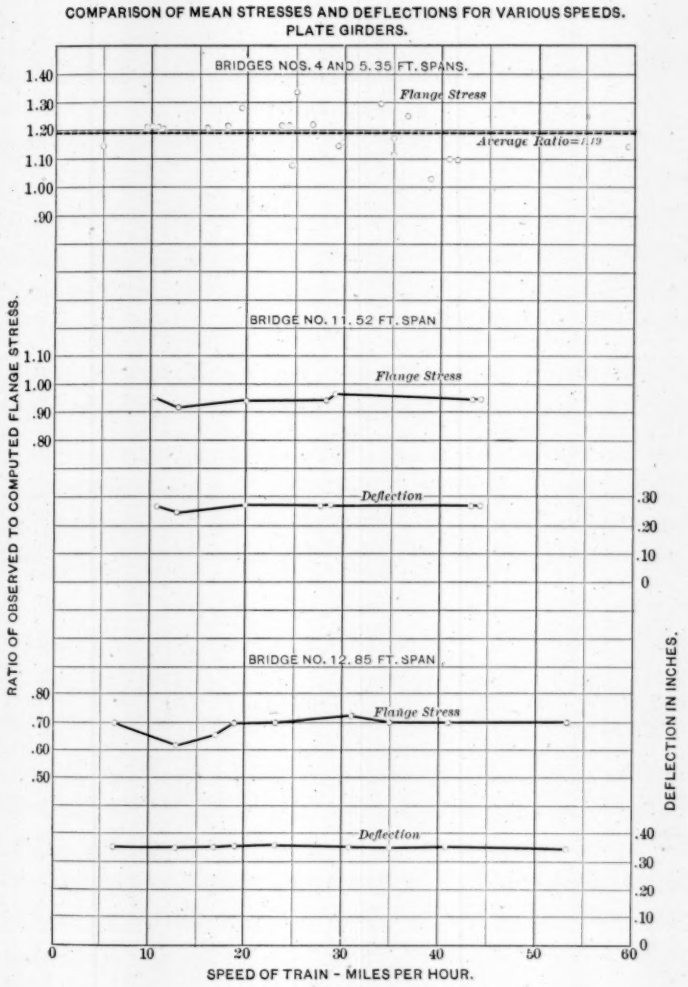

COMPARISON OF MEAN STRESSES AND DEFLECTIONS FOR VARIOUS SPEEDS.
PLATE GIRDERS.
BRIDGES NOS. 4 AND 5. 35 FT. SPANS.
Flange Stress
Average Ratio=1.19
BRIDGE NO. 11. 52 FT. SPAN
Flange Stress
Deflection
BRIDGE NO. 12. 85 FT. SPAN
Flange Stress
Deflection
RATIO OF OBSERVED TO COMPUTED FLANGE STRESS.
DEFLECTION IN INCHES.
SPEED OF TRAIN - MILES PER HOUR.

FIG. 12.

the computed, on account of the secondary stresses; but by plotting the observations in this way it was thought that any possible effect of speed would be noticed. In the tests on Bridges Nos. 4 and 5 (Fig. 12) and 16 (Fig. 13) the trains were all different. In the other cases the observations for each bridge were made by a single train, and, although ratios of observed to computed stresses are plotted here also, the computed stresses are the same for all speeds, and the observed values are directly comparable. The deflections are likewise comparable, and are therefore plotted. As to camber, the practice of the Chicago, Milwaukee and St. Paul Railway Company is to omit all camber in girders and to put about the usual amount in trusses. There were no irregularities in the track sufficiently great to be noticeable.

As was to be expected, the points in the diagrams for Bridges Nos. 4, 5 and 16 are somewhat scattered, but the points in all the others show much regularity in arrangement. It is quite evident that the diagrams fail to show in any case any general increase in stress or deflection with increase in speed. On the contrary, they show in nearly every case very constant values at all speeds, and, considering the uniformity of the results and the conditions under which the experiments were made, the conclusion seems to be warranted that the effect of speed alone on bridges of spans exceeding 40 ft. in length is of no consequence whatever. For shorter spans the experiments are not sufficiently accurate to be of much value in a discussion of this question.

Deductions from Theory.—If a load in passing over a bridge causes the structure to deflect so that the curvature of the track is concave upward, the pressure of the load on the bridge will be more than its weight by the amount of the centrifugal force developed. If v = velocity along the curved path (assumed equal to the horizontal velocity); M = mass; and r = radius of curvature of the path of the load (not necessarily of the track), then the centrifugal force, F, $= \frac{Mv^2}{r}$.

Take, for example, the case of a single load moving over a beam of constant cross-section. When the movement is slow, the equation of the path of the moving body is:

$$y = y_o \left(1 - \frac{x}{a}\right)^2 \left(1 + \frac{x}{a}\right)^2,$$

where x and y are the usual coordinates, with origin at center; a = half span; and y_o = central deflection with load at center. The sharpest curvature of this path is at the center, where the radius of curvature,

COMPARISON OF MEAN STRESSES AND DEFLECTIONS FOR VARIOUS SPEEDS.
TRUSSES.

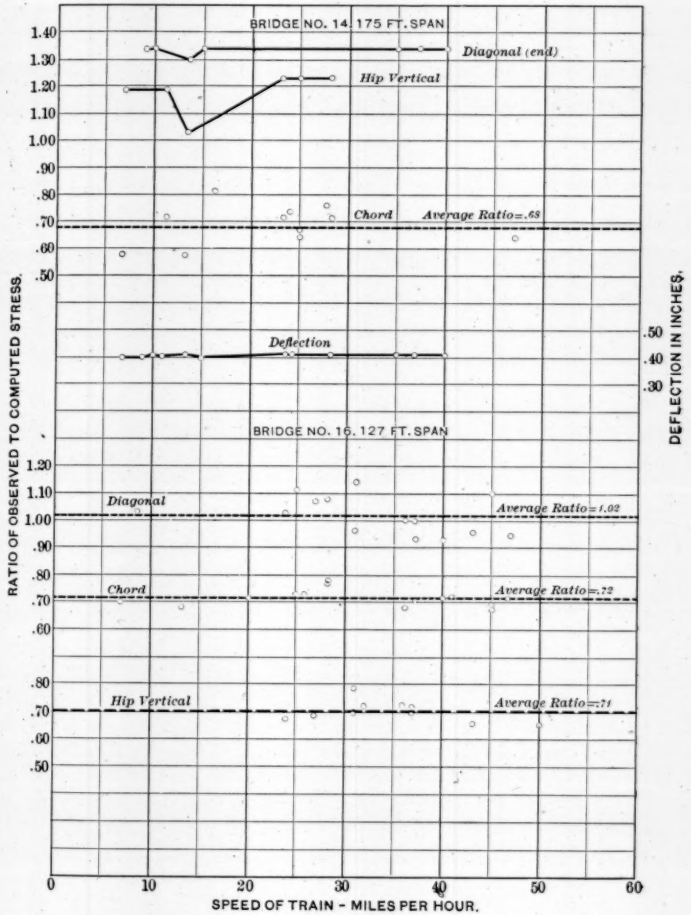


FIG. 13.

$r, = \frac{a^2}{4\ y_o}$, whence, at this point, the centrifugal force is, approximately, $F = \frac{4\ M\ v^2\ y_o}{a^2}$. If $y_o = \frac{1}{2400}$ of span, and $v = 90$ ft. per second, or about 60 miles per hour, then $F = \frac{0.84}{a} P$, in which P = weight of moving body.

Thus, for a 25-ft. span, $F = \frac{0.84}{12.5} P = 7\%$ of weight; for a 50-ft. span, $F = 3.3\%$ of weight. As the actual deflection is somewhat increased by the centrifugal force, and hence the sharpness of curvature likewise increased, the effect is really somewhat greater than the above.

Dr. H. Zimmermann has treated this problem rigidly,* and a formula closely approximate to his exact formula (practically correct for values of $\frac{g\ a^2}{4\ v^2\ y_o} > 10$) is:

$$F = P \frac{1}{\frac{g\ a^2}{4\ v^2\ y_o} - 3},$$

or, with $y_o = \frac{1}{2400}$ of span, and $v = 90$ ft. per second, it becomes

$$F = P \frac{1}{\frac{a}{0.84} - 3},$$

a formula practically exact for values of a greater than about 8 ft.

This formula gives for the above cases, 8.7% and 3.7%, respectively. For a 15-ft. span, the increase would be 16%, with the same assumption as to the ratio of deflection to span length. For such short spans, however, the deflection would not, as a rule, be so great as $\frac{1}{2400}$ of the span, so that no higher value would be likely to be obtained than for a 25-ft. span. For a 100-ft. span, $F = 1.7$ per cent. The effect of a uniform load would be less than that of a concentrated load, as the average curvature is in that case much less.

By the above approximate methods it is shown that theory, as well as the results of experiments, indicates that the effect of speed alone is of no practical importance unless it be for very short spans. For such spans experimental data are not available. The best formula known to the author is that by Zimmerman. The effect of camber is to reduce the centrifugal force, but the small value of the force indicates that the benefit derived in this direction from cambering ordinary spans is small.

*"Die Schwingungen eines Trägers mit bewegter Last," by Dr. H. Zimmermann, Berlin, 1896.

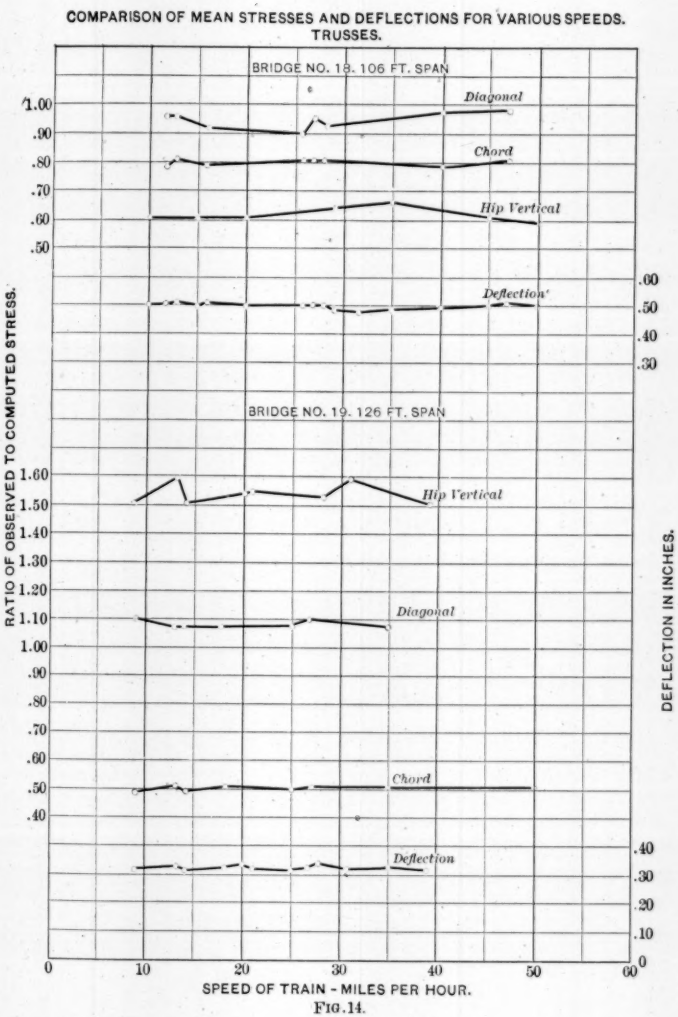

COMPARISON OF MEAN STRESSES AND DEFLECTIONS FOR VARIOUS SPEEDS.
TRUSSES.
BRIDGE NO. 18. 106 FT. SPAN
Diagonal
Chord
Hip Vertical
Deflection
BRIDGE NO. 19. 126 FT. SPAN
Hip Vertical
Diagonal
Chord
Deflection
RATIO OF OBSERVED TO COMPUTED STRESS.
DEFLECTION IN INCHES.
SPEED OF TRAIN - MILES PER HOUR.

FIG. 14.

Vibrations in Girders and Trusses.

It will be convenient in discussing the subject of vibration, to treat the girders and trusses separately. As a rule the girders were of too short span-length to show cumulative vibration, the causes producing vibration acting but a few times, and usually at a much slower rate than the natural rate of vibration of the structure. In the truss bridges, however, conditions were often favorable for cumulative vibration, and this point is considered in that part of the discussion.

Vibrations in Girders.—The typical curves on page 417, already described, show that the feature of the tests which is of chief interest and importance is that of the vibrations caused by the locomotive. The results of all tests of any value have therefore been presented in Table No. 3, with particular reference to this point.

In the column headed deflection, and in the column containing values of stress per square inch, the several quantities were found by measuring the mean ordinates of the respective vibratory curves at the point where the deflection, or stress, was a maximum under the locomotive. The "percentage added by vibration" is the ratio of one-half the amplitude of vibration to the mean ordinate as thus determined. In the short spans the "vibrations per second" were not very accurately determined, as often there would be but one or two vibrations during the passage of the locomotive. The other columns are self explanatory. Where the records are of doubtful value, owing to the inertia of the instruments, a question mark is placed after the figures; but figures have not been given except where a careful inspection of the curves has satisfied the author that the actual vibrations were a large percentage of those recorded. Records of deflection are poor for many of the experiments on spans up to 45 ft. in length. The experiments on each bridge are arranged in the order of increasing speed of trains. The condition of track was good on all bridges.

The percentage of increase in the flange stress should agree in general with that in the deflection. An examination of Table No. 3 shows that, while the agreement is in many cases not very good for the shorter spans, it is good for the longer spans, and on the whole, considering the great difference between the two kinds of instruments used, is sufficient to show that the results are entirely reliable as to general indications. Several experiments have been omitted from the table,

the vibrations being too small to be of any consequence. All such tests, however, are accounted for in the column of remarks.

The principal points brought out by the data in Table No. 3 are:

(1) The increase in deflection, or flange stress, due to vibration is about the same in all spans from 25 to 55 ft. in length; and, neglecting two or three doubtful cases, has a maximum value of about 50% for speeds of 40 or 50 miles per hour. For spans of 60, 70 and 85 ft. in length, the maximum percentages are 22, 28 and 17 respectively.

(2) Speeds greater than about 20 miles per hour for short spans, and 35 miles per hour for long spans, almost invariably caused large vibrations.

(3) The rates of vibration of the bridges for high speeds and large amplitudes agree approximately with the rates of revolution of the locomotive drivers, and show quite clearly that the chief influence in producing vibration is the effect of the locomotive counterweights.

(4) There are undoubtedly other causes of vibration, such as roughness of track, flat wheels, etc. This is shown to some extent by the results from comparatively slow speeds, where the vibrations produced are much more rapid than the rate of rotation of the drivers. The presence of an iron floor system in the through girders, Nos. 10 and 12, appears to make no difference in the vibration. The panel length in both cases is about 14 ft., a very favorable length for vibration in Bridge No. 12, where a locomotive with 56-in. drivers was used.

(5) Different types of locomotives, and locomotives differently balanced, will undoubtedly produce different amounts of vibration, but in these tests there was little variety in locomotives or in counterbalancing. The ten-wheel engines appear to have caused more vibration in some cases than the American type, although the difference is hardly enough to warrant a general statement. The tests on Bridges Nos. 11 and 12 were all made with a ten-wheel engine with small drivers, but the resulting vibrations are rather low.

(6) As to a law of variation of vibration with speed, the experiments were not numerous enough to determine this point. In a general way vibration increases with speed, but apparently not as rapidly as the square of the speed, as the theory of centrifugal force would suggest. An attempt was made to discover any possible regularity in variation by plotting the experiments, but the accidental variations were too great to make the diagrams of any value.

(7) In the through girders, Nos. 10 and 12, considerable vibration was caused in some cases by loaded freight cars, although the panel length of 14 ft. is not favorable to such action (see Fig. 5).

Vibrations in Trusses.—Table No. 4 gives the principal data from the experiments on trusses, arranged as before, in the order of increasing speed for each bridge. As has already been explained, the extensometers were used on the lower chord (usually at the center), the diagonal nearest the center, and the hip vertical, the various members tested being indicated in the small diagrams in the table by heavy lines. Stresses, and percentages of increase due to vibration, are given for these members, together with corresponding values for the deflections. Where the vibrations were of any consequence, they agreed in rate for the deflection, chord stress, and stress in the diagonals, and also, usually, for the hip vertical.

In some cases the relative vibration was greatest before or after the maximum deflection occurred, and for some of these cases two values for vibration are given, one for the maximum vibration, and the other for vibration at time of maximum deflection.

The effect of vibration will be discussed first with reference to the deflection only, the relation between stress and deflection being taken up later. The salient points in the tests on each structure will be briefly noted.

Bridge No. 13 is one of the worst for vibrations. Under the locomotive there are many values above 15%, the highest being 20%, and highly cumulative (see Fig. 7). These large vibrations occur for speeds of 27 to 32 miles per hour, and are all for freight trains. The freight locomotives are much larger than the passenger, and are ten-wheelers, which facts may account for the smaller vibrations for speeds above 32; but it is more likely that this is due to the fact that the higher speeds do not accord so well with the natural rate of vibration for the loaded bridge. Fig. 6 is a record from a passenger train and shows much less vibration than Fig. 7, although the speed is much higher. Several curves from this bridge show large cumulative vibrations from loaded cars, but in no case is the total deflection equal to that produced by the locomotive. The conditions in this bridge are very favorable to vibration. The panel length is about equal to the circumference of a locomotive wheel, and, furthermore, the stringers are shallow and the structure is rather light and flexible. Passenger cars and empty freight cars cause no vibration.

Bridge No. 14 is a modern structure. The trains at this place were very light, as were also the locomotives. The maximum vibration was caused by a passenger engine at a speed of 47 miles per hour. Speeds below 15 miles per hour did not cause noticeable vibration.

Bridge No. 15 is a light structure. Speeds below 18 miles per hour gave little vibration. The maximum for locomotives is about 20% for a speed of 42, and that for cars is 28% at 28 miles per hour. In experiment No. 145, with a speed of 45, the vibration is greatest before the locomotive reaches mid-span, showing that this speed was probably too high for maximum cumulative effect. This bridge has a wooden floor, and a panel length of 17 ft. 4 ins., conditions very favorable for vibrations, both for cars and locomotives.

In the tests on Bridge No. 16, in a large number of cases, there was considerable vibration caused by the engine. Few records show much effect of cars, but there were very few freight trains observed here at speeds above 20 miles per hour. The floor system is not as flexible as in some of the other bridges in which wooden stringers are used, but the panel length is favorable for vibration, and this may partially account for the comparatively high percentage for so large a number of tests. The records from this structure show that speeds may vary within quite a large range, and still have conditions favorable for vibrations.

Bridge No. 17 is a thoroughly modern one with stiff lower chords and small deflection, but it shows fully as great vibrations from passenger engines as the lighter bridges. Freight trains were probably too slow here to cause much vibration.

Bridges Nos. 18 and 19 are also modern structures, No. 19 being very similar to No. 17. They were experimented upon by the special train before mentioned. The 106-ft. span shows 23% vibration under the locomotive for a speed of 25 miles per hour, and less for higher speeds; but the loaded cars produced much vibration at the high speeds. The panel length in this case is nearly equal to the driver circumference, and this may have influenced the vibration due to the engine. The vibration due to the cars cannot, however, be attributed to the stringers, for the rate of vibration implies a distance passed over of about 25 ft. for each vibration, while the panel length is 17 ft. 8 ins. It possibly may have been due to rough track near the bridge.

With Bridge No. 20 few experiments were made at sufficiently high speeds to develop vibrations, none at a speed of less than 26 miles per

hour causing appreciable disturbances, although ten such tests were made. The panel length of 25 ft. is quite unfavorable for vibration, the locomotive used having 56-in. drivers.

In the next four cases most of the records of the deflectometer were obliterated by dampness, and the records of the chord stresses must be used in lieu of anything better.

There is in the first three of these bridges slightly less than the usual amount of vibration, but the difference is not greater than would be accounted for by the lower speeds here observed. It was expected that No. 21 would have considerably less vibration than No. 22, owing to its much greater rigidity, but the tests failed to show much difference. In none of these three bridges was the panel length favorable to vibration. In the last case no observations were made on the chord, and but three of the records of the deflectometer are available. The results from the diagonal indicate rather high vibration, amounting to 26% in one case. The speeds are somewhat higher here than in the other three cases.

Cumulative Vibration.—In all cases where large vibrations were produced in the trusses tested, they were, without doubt, more or less cumulative, and highly so in the worst cases; as, for example, in the test illustrated in Fig. 7. The chief cause of vibration under the locomotive was undoubtedly the counterweights, the effect of which was increased in some cases by the deflection of the stringers; and the results show that cumulative vibration from this cause can occur for a comparatively wide range of speed, the rate of vibration increasing with the speed. True cumulative vibration occurs only when the impulses are timed to agree with the rate of vibration of the structure; but it is to be noted that as the locomotive passes upon a bridge the natural rate of vibration changes gradually from that of the empty bridge to that of a fully loaded one. The speed of the train may, therefore, vary between quite wide limits and yet cause cumulative vibration. At the same time, this change in rate of vibration of the bridge limits to a considerable extent the duration and intensity of the vibrations. It is quite noticeable, for example, in Bridge No. 13, that at the higher speeds the vibration was usually of the greatest relative intensity before the locomotive reached the center of the bridge, while with the slower speeds it was greatest with the bridge fully loaded (see Figs. 6 and 7). Other cases of high vibration occurring before the maximum deflection was reached are noted in the table (see also Fig. 8). In the case of vibrations produced by a uniformly loaded

train, the rate remains constant, and the vibrations often become highly cumulative. Speeds too low for cumulative effect will cause non-cumulative vibration, as in plate girders, which increases in intensity with decrease in span. Speeds too high for cumulative effect will cause shocks to stringers and other primary members, which will be absorbed before reaching the main truss.

Professor Robinson has thoroughly discussed the subject of cumulative vibration,* and in his paper derives an approximate formula for the time of vibration of loaded bridges. His formula for a uniformly distributed load is :

$$t = 2\pi\sqrt{\frac{2\ W\ d}{3\ g\ P}}$$

in which t = time for one complete vibration; W = total load, including weight of bridge; and d = central deflection produced by any distributed load P. He has further shown the effect of car and locomotive springs in reducing the time of vibration of bridge and train, and presents a formula taking account of this effect. The author has made calculations by this formula, and also by a somewhat different one of his own, but with very unsatisfactory results, especially in the case of the locomotive. Calculations based on constants obtained from the railroad company, would call for a rate of vibration of the locomotive on its springs of about 2 per second, whereas the rate was found by actual observation to be between 4 and 5. It, however, can be easily shown experimentally that a combination of two elastic systems will vibrate as a single system at a rate somewhat slower than that of the most slowly vibrating component. Without attempting to take account of this effect of the springs of the locomotive, or cars, the author has calculated, by the above formula, the rate of vibration for each truss when loaded with the heaviest locomotive actually observed in each case, followed by a loaded freight train weighing 1 600 lbs. per lineal foot (about the actual weight). These values are given in the summary of tests on trusses.

Summary of Tests on Trusses.—In Table No. 5 is presented a summary of the principal data of the tests on trusses. Detailed information concerning the trusses is given in Table No. 2. In the third column of the table are given the ratios of deflection to span-length, under the average heavy train observed, to serve as a measure of the flexibility of

* *Transactions*, Am. Soc. C. E., Vol. xvi, p. 42.

the trusses under the actual traffic. As the locomotives varied considerably in weight, these figures do not show the relative flexibility of the bridges under like conditions.

The most noteworthy feature in these results is, perhaps, the comparative uniformity in the maximum percentages added to deflection by vibration. A reference to the details of the tests in Table No. 4, shows that the rate of vibration of the structures agrees quite closely with that of the revolution of the drivers, indicating that, as in plate girders, the chief cause of vibration is the unbalanced wheels. With the exception of Bridge No. 23, the only ones in which the vibration varies much from an average value are Nos. 14 and 19. These are among the stiffest in the list, but there are others having high vibration which are equally stiff, and it can hardly be concluded that stiffness is the cause of the small amount of vibration in these two cases. In fact, there is not enough variation in the results to enable conclusions to be drawn respecting the influence of any of the varying conditions.

These tests do indicate, however, that the vibration in almost any truss of the span-lengths here considered, is very likely to be as high as 20%, but that it is not likely to greatly exceed this percentage under ordinary traffic. Considering the large number of experiments and the variety of trusses, it would seem that the maximum might reasonably be placed at 25% for trusses of from 100 to 200 ft. in span-length, and perhaps for longer spans, as the tests do not indicate any tendency to an increase in vibration with increase in span.

The minimum speed causing vibration exceeding 10% of the static deflection, varies from about 20 to about 36 miles per hour, averaging, say 30. In two cases, Nos. 13 and 18, both flexible bridges, the maximum speed causing excessive vibration, was reached and passed. In all other cases the highest observed speed caused more than 10% vibration. The calculated rates of vibration are evidently too high. In Nos. 13 and 18 they correspond with the maximum velocity of train that causes excessive vibration, while in Nos. 22 and 23 they are less.

As regards vibration caused by cars, very few cases were noted for each bridge in which this was excessive, and, on account of the small number of loaded freight trains passing at high speeds, the records for Bridges Nos. 14, 17 and 20 are not of great value in this connection. It is noteworthy that the high percentages for cars are all for bridges in which the panel-length is very nearly equal to half a car length. Bridge

No. 22 is a very flexible structure, yet the vibrations produced by trains are invariably small.

Calculated Effect of Counterweights.—The practice of the Chicago, Milwaukee and St. Paul Railway in counterbalancing is to counterbalance for the revolving parts and for two-thirds of the weight of the reciprocating parts, the weight for the reciprocating parts being equally distributed among the drivers. Weights of the reciprocating parts have been obtained for two or three locomotives of each important class of engines involved in the tests on this company's bridges, and average values are herewith given, together with the amount of excess counterweight for each wheel, in terms of weight applied at crank-pin center. The class of locomotive is here indicated by its weight, which will enable it to be identified in the tables of tests. The centrifugal forces of these excess weights at speeds of 40 and 50 miles per hour have been computed, and are given in the following table in terms of percentage of weight on the driver. Account is taken of the fact that the counterweights are 90° apart and the resultant effect is here given. Nos. 5 and 6 are ten-wheel engines.

No.	Weight.	Diameter of drivers.	Stroke.	Weight on drivers.	Excess weight on each wheel.	Centrifugal Force (per cent).	
						40 miles per hour.	50 miles per hour.
	Lbs.	Ins.	Ins.	Lbs.	Lbs.		
1.....	141 000	69	24	50 000	180	13	21
2.....	154 000	62	24	54 000	152	13	21
3.....	146 150	62	24	55 400	179	16	24
4.....	178 200	68	24	64 100	203	12	19
5.....	189 100	62	26	84 500	168	16	24
6.....	170 600	56	26	76 700	155	19	30

For spans less than about 35 ft. in length the deflection caused by a locomotive is due almost entirely to the weight on the drivers at the time when it reaches its maximum value; and for such bridges these calculations would indicate an increase in deflection from the centrifugal force of the counterweights, amounting to from 20 to 30% for speeds of 50 miles per hour. For spans 50 ft. long the entire locomotive is on the bridge when the maximum deflection takes place, but the drivers still cause about two-thirds of this, and the calculable value of the effect of the counterweights would therefore be approximately 15% of the static deflection. For a 100-ft. span the values would be from one-third to

one-half of those given in the table, or from 7 to 15%, for velocities of 50 miles per hour. In all this computation, cumulative action has been neglected. Such action, undoubtedly, increases greatly the vibration in the longer spans.

The above-computed values are considerably less than the observed, but they suffice to show that, for short spans at least, vibration is certain to be large so long as locomotives are used which cannot be perfectly balanced. The excess of the observed vibration over the computed can, undoubtedly, be accounted for in many cases by the fact that the locomotive is vibrating on its springs as it passes upon the bridge. The author has endeavored on two occasions to learn, if possible, by observation from the locomotive, if such vibration took place. In both cases very considerable vibration was noted frequently as occurring at a rate of about 4 per second, and when it was most noticeable it agreed almost exactly with the rate of revolution of the drivers. Small bridges were crossed in some instances while this vibration was quite marked, and a test would, no doubt, have shown the effects of this.

Relation of Stresses to Deflections.

Chords.—An examination of the table will show that the percentage of increase in chord stress is usually about the same as in deflection. In a few cases there is considerable difference, sometimes the chord stress being the greater and sometimes the deflection.

Diagonals.—Here, again, in many cases the variation in stress agrees with that in the deflection, but on the average the increase in the former is considerably the higher. Considering only the effect of the locomotive, and discarding those tests in which the train moved in the wrong direction to cause a large stress in the diagonal, it will be found that the maximum percentage of increase for the diagonal is 30%, compared with 19% for the deflection in the same test. As the maximum stress in the center diagonal occurs before the maximum deflection, the increase in its stress due to vibration depends upon the vibration occurring slightly earlier than that which determines the increase in deflection as usually recorded (see Tests Nos. 33 and 27, on Bridge No. 13).

Hip Verticals.—The increase in stress in the hip vertical from vibration is in most cases much higher than in deflection, the maximum values varying between 15 and 47%, if one doubtful value of 72% is omitted. If most weight is given to the figures obtained from a large

number of experiments on the same bridge, an average maximum value would be about 40%, or approximately the same as found in plate girders of 40 or 50-ft. span lengths.

From the theoretical standpoint these results are about as would be expected. The action of a locomotive on a bridge when cumulative vibration takes place is two-fold: First, there is the local effect of each impulse from the drivers, and, second, the effect on all parts of the structure due to vibration. The effect on the chord members is mainly from the latter cause; that on diagonals from both, while the hip verticals and floor system near the end of the bridge would feel mainly the local effect. Stringers and floor beams near the center of the bridge would feel both the effect of the extra pressure due to vibration, and the impact of the drivers. No experiments were made on stringers or floor beams, but assuming them to receive the same impact as short girders, the total effect on these members would be something like 75 per cent. This may in reality be somewhat reduced by the more or less elastic supports of these girders.

Secondary Stresses and Computed Deflections.

As incidental to the main object of the tests, there were many cases where secondary stresses were determined by using both extensometers on the same member. As some of the results may be of interest, indicating as they do the possible magnitude of these stresses, they will be here given. In the plate girders, for example, the observed stresses varied from the computed as follows:

For 25-Ft. Span.

Top outside flange, observed value = 130% of computed.
Bottom " " " = 180% "
" inside " " = 45% "

For 30-Ft. Span.

Top outside flange, observed value = 125% of computed.
Bottom " " " = 170% "
" inside " " = 70% "

For 45-ft. spans and longer the flange stresses agreed very well with the computed, except in the through girders, where the lower flange stress was only about 70% of the computed value.

In general, the different bars of the same chord member had nearly the same stress, but all had more or less bending stress near the pin. This was not great for well-proportioned eye-bars, but was high for some very light bars on Bridge No. 15; and in the stiff chords of Bridge No. 17 the upper flange had, near the pin, only 75% as great a stress as the lower, while in a similar bridge, No. 19, the upper flange had less than 60% of the lower.

Diagonal bars pulled about equally, except in Bridges Nos. 13 and 16, where in one case the outside bar received 65%, and in the other only 50%, as much as the inside.

Hip verticals, where made of bars, were about equally stressed. In one case of stiff members the outside flange received 40% as much as the inside, and in another case 70 per cent.

In those bridges having an iron floor system the observed chord stresses were always from 20 to 30% less than the computed (see Figs. 2 and 3), whereas the observed diagonal stresses averaged about the same as the computed. The cause of this small chord stress was, undoubtedly, the action of the stringers as tension members. To find what effect this tension had on the floor beams, a few experiments were made on the end floor beam of one bridge. A stress as high as 3 000 lbs. per square inch was observed in the flanges, due to horizontal bending, this value being two-thirds as great as the stress due to the vertical load.

In the lighter bridges, and in the long diagonals of nearly all the bridges, considerable vibration occurred in the individual bars, causing high stresses in some cases, and being objectionable in general. This was not observed in the chord bars of the heavier structures.

It is doubtless true that secondary stresses are often much greater than stresses due to the vibration of the structure as a whole; and it is an open question whether more attention should not be paid to this feature. Experiments with the apparatus herein described can readily be made, and, if thoroughly carried out for a few well-designed trusses of any given form, would give a good idea of the amount of these secondary stresses. Theoretical investigations are for many cases quite practicable, but for others are too complicated for ready use, and actual tests are much more instructive. A large railway company could well afford to possess a few of the instruments such as described in this paper, as tests could easily be made which would be of much assistance in the development of standard designs.

Deflections have been computed for Bridges Nos. 18 and 19 by the formula involving the distortions of each member, $D = \Sigma \frac{p\,u\,l}{E}$. The results are, for No. 18, $D = 0.54$ in., and for No. 19, $D = 0.37$ in.; or, making approximate allowance for reduced chord stress on account of the action of the stringers, these values become 0.50 in. and 0.35 in., respectively. The observed values averaged about 0.49 in. and 0.33 in., respectively.

Results of Other Experiments.

The first published report of experiments on bridges under moving loads, that has come to the author's notice, is that of some tests made by Professor Fraenkel.* They were made with instruments similar to those described in this paper, but the extensometer as then constructed was not very reliable. The tests are of value chiefly in showing the amount of the secondary stresses in small riveted girders, but a few experiments were made which bring out the effects of high speeds. Deflection tests on four small girders showed an increase of from 2 to 48% in the deflection (including the effect of vibration) over that occurring at slow speeds. In a truss of 36.37 m. span the chord stress was not increased by an increase in speed; but there was considerable increase shown in web members and in the floor system.

The first really important series of tests are the well-known experiments made by Professor Robinson,† to which reference has already been made. The results of his tests on five bridges of from 141 to 156-ft. span-lengths show percentages of vibration for the locomotive varying from 19 to 28.6. One of these spans was a Whipple truss, while all others were Pratt trusses. All had panel-lengths approximately equal to the locomotive driver circumference; the stringers were in three cases wooden, and in the other cases iron, of a depth of from 24 to 26 ins. In two other bridges of 128-ft. spans, the vibration was 11.4% and 15.9 per cent. In these the rails rested on wooden floor-beams supported directly by the lower chord. Still another bridge, a very stiff, double-track span of 154 ft. 6 ins., with a panel length of 19 ft. $3\frac{3}{4}$ ins., was reported as having very little vibration. In all these tests the maximum vibration from cars was 50 per cent.

These experiments by Professor Robinson agree very well with those made by the author, except that for cars the maximum vibration

* *Der Civilingenieur*, 1881–1884 and 1887.

† *Transactions*, Am. Soc. C. E., Vol. xvi, p. 42.

observed by the latter was somewhat less. In no case, however, in any of the experiments, was the maximum deflection from a train as great as that from the locomotive.

A test of a riveted bridge of 35-m. span is reported by M. Cuenot.* The Manet apparatus was used, and it is stated that the stresses in the various members averaged about 22% higher under a speed of 25 km. per hour than at a low speed, or under static load. As the instrument is not one which gives a continuous record, it is to be supposed that this increase in stress includes all effects of vibration. The detailed figures show much irregularity in the percentages for the various members.

A series of experiments which will undoubtedly be of great value when completed is that now being carried on by M. Rabut on the Western Railways of France. M. Rabut reports his tests to show† that in a plate girder of 60-ft. span, with a static deflection of $\frac{3}{8}$ in., vibration will not increase the deflection by more than one-tenth. He also finds the same relative increase in stress as in deflection. Secondary stresses are found to be large. From some previous work done by him on eight bridges, of spans from 4 m. to 62 m. in length,‡ he comes to the following conclusions: At high speeds vibration is governed by wheel-base, cars, and panel-lengths; it increases with the speed. In large spans this vibration is only a small fraction of the static deflection. Excess of dynamic over static deflection is very small with main girders of long spans; it is sensible for beams and main girders for short spans, and considerable for stringers, especially near abutments. The time of vibration is independent of speed. Increase in speed scarcely affects mean deflection, but increases vibration, although this increase is, for large spans, only a small percentage of the deflection. As regards increase in mean deflection from increased speed, these results agree with the author's experiments, but the effects of vibration appear to be much less marked in the experiments of M. Rabut.

Professor M. A. Howe has described§ some observations of vertical and lateral deflection made on two bridges of 122 and 164-ft. span lengths. Very little vibration was observed in these experiments, the highest for trains being 7.5%, and for a single locomotive 10 per cent. The speeds were, however, all low, nearly all being below 25 miles per

* *Annales des Ponts et Chaussées*, July, 1891, p. 5.

† *Engineering*, August 13th, 1897, p. 183.

‡ *Le Génie Civil*, Vol. xxii, 1892, p. 88; abstracted in *Min. Proc. Inst. C. E.*, Vol. cxii, p. 391.

§ *Journal of the Association of Engineering Societies*, Vol. xiv, 1895, p. 513.

hour, which, from the author's tests, appears to be too low to develop much vibration.

A number of measurements of stresses in individual members were made on three bridges, in 1893, by Messrs. Hankenson and Ledger, using for the work the extensometer already mentioned.* Their experiments were mainly on hip verticals, but some records were obtained from other members. The curves show some vibration similar to that found by the author, and indicating excess of dynamic over static stress, but the increase appears to be considerably less. The diagrams from the lower chord show considerable vibration of the structure as a whole, but the record is mostly masked by the very rapid vibrations of the instrument, which were accompanied by a jarring sensation, as noted by the experimenters. Their instrument was evidently very sensitive to these rapid vibrations which, however, may have been really very small, as the author is inclined to believe was the case. The records from the diagonal indicated as much as 20% increase in stress due to vibration of the structure.

The latest mention of experiments on bridges is in the recent paper by Mr. Stone.† In this paper the author bases his coefficients for "immediate effect" entirely on results of tests on bridges in India. Average percentages from these results are given in the form of a curve on page 372. From this it appears that the experiments referred to showed an increase due to impact of from 50% for very short spans, to about 14% for spans in which the ratio of fixed to moving load is one-fourth, and about 7% where this ratio is one-half. While it is probable that these values for short spans are none too high and agree very well with the results of the author's experiments, the coefficients for the longer spans appear to be much too low for American practice, as indicated both by Professor Robinson's experiments, and those herein described. It would be very desirable to know more in detail of the nature of the tests referred to by Mr. Stone.

Conclusions.

The following is a summary of the principal results and conclusions which have already been mentioned:

(1) Speeds less than about 25 miles per hour are not likely to result in much vibration.

* *Engineering News*, May 9th, 1895, p. 300.

† "The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel," by E. Herbert Stone, M. Am. Soc. C. E.; *Proceedings*, Am. Soc. C. E., May, 1898, p. 364.

(2) The increase in deflection due to vibrations, caused by locomotives running at speeds of 40 to 50 miles per hour, is likely to be 40 or 50% for girder spans of less than 50 ft. in length.

(3) This percentage decreases rapidly for longer spans, becoming about 25% as a maximum for 75-ft. spans.

(4) Owing to cumulative effect, the percentage is likely to be a maximum of 20 or 25, for spans from 75 ft. to 150 ft., or more, in length, but the experiments indicate no increase in percentage for increase in span.

(5) The relative increase in chord stress is about the same as in deflection; that in center diagonal is somewhat more than in the deflection; and in hip vertical it corresponds more nearly to that in girders of 40 to 50-ft. span-lengths.

(6) The effect of speed of application of the load on mean deflection was of no consequence in the spans tested (the increase in deflection from live load being due to vibration), although theory points to an appreciable increase from this cause in very short spans without camber.

(7) Secondary stresses are likely to be high in small girders with shelf-angles, and in some parts of trusses, and the discrepancy between observed and computed stress may be greater from this cause than from the dynamic effect of moving loads.

In conclusion, the author desires to acknowledge his great indebtedness to the various officials of the Chicago, Milwaukee and St. Paul Railway and of the Union Pacific Railroad, who kindly furnished facilities and information which have enabled these experiments to be carried out. Of those connected with the former road, special thanks are due to Onward Bates, M. Am. Soc. C. E., Engineer and Superintendent of Bridges and Buildings, for his great assistance and kindly interest throughout the entire work; to J. N. Barr, M. Am. Soc. C. E., Superintendent of Motive Power, and Mr. A. E. Manchester, Assistant Superintendent of Motive Power, for much information regarding weights of locomotives and cars; and to Division Superintendents Messrs. J. W. Stapelton and C. A. Cosgrave for special facilities in the way of trains. To George H. Pegram, M. Am. Soc. C. E., former Chief Engineer, and Mr. J. B. Berry, now Chief Engineer of the Union Pacific Railroad Company, the author is indebted for facilities and data in connection with the tests made on the four bridges of that road.

The author is also greatly indebted to A. D. Stewart, M. E., for many valuable suggestions and for his assistance in carrying out the experiments.

TABLE No. 1.—Dimensions of Girder Bridges.

Reference No.	R. R. Co.'s No.	No. of spans.	Length of span, out to out.	Depth.	Distance of shelf-angles below upper flange.	Weight of one span, in pounds.	Date of construction.	Remarks.
1	Z 448½	1	25′ 0″	1′ 6½″	1′ 0″	13 300	1892	Tested east span. Each span is composed of four girders.
2	L 98	3	25′ 0″	1′ 6¼″	1′ 0″	13 400	1894	
3	L 254	1	30′ 0″	2′ 6″	1′ 3″	12 900	1894	
4	L 92	2	35′ 0″	3′ 0″	1′ 2½″	16 400	1894	Tested east span.
5	Z 450	2	35′ 0″	3′ 0″	1′ 2½″	16 600	1892	" "
6	Z 452	2	45′ 0″	3′ 6″	1′ 2½″	25 700	1892	" "
7	L 204	1	45′ 0″	3′ 6″	1′ 2¼″	26 600	1895	
8	Z 318	1	55′ 0″	4′ 6″	2′ 1½″	40 500	1895	
9	L 260	3	60′ 0″	5′ 3″	1′ 2½″	40 900	1894	Tested west span.
10	Z 484	3	70′ 0″	6′ 6″	Five panels; 24-in. beams; 19-in. stringers.	150 500	1890	Double track, with three girders. West span tested.
11	R 6	3	52′ 0″	4′ 4″	2′ 2½″	37 150	1896	Skew bridge. West span tested.
12	R 36	1	85′ 0″	7′ 0½″	Six panels; 30-in. beams; 22-in. stringers.	116 800	1896	

TABLE No. 2.—Dimensions of Truss Bridges.

Reference number.	Railroad company's number.	Number of spans.	Length of span, center to center.	Number of panels.	Panel length.	Depth.	Floor beams.	Stringers.	Hip verticals.	Weight of one span, in pounds.	Date of construction.	Remarks.
13	Z 312	5	158′ 3″	{ 8 / 1 }	{ 17′ 9″ / 16′ 3″ }	25′ 0″	30-in.	{ Four 15-in. / I-beams. }	Two 8-in. ⊔'s	212 500	1883	Skew bridge. Tested west span.
14	K 96	1	175′ 0″	8	21′ 10½″	29′ 0″	38½-in.	27-in.	Two 6 x ⅞-in.	285 400	1896	
15	L 206	2	104′ 0″	6	17′ 4″	23′ 0″	24-in.	Eight 8 x 16-in.	Two 2½ x 1-in.	100 000±	1879	Trestle approach at west end. West span tested.
16	L 106	2	127′ 9″	7	18′ 3″	24′ 0″	27-in.	{ Four 15-in. / I-beams. }	Two 6-in. ⊔'s	142 900	1886	East span tested. Trestle approach at west end.
17	L 90	1	123′ 6″	6	20′ 7″	27′ 0″	42-in.	Four 30-in.	Two 4 x 1¼-in.	179 500	1893	Lower chord stiff. Trestle approach at each end.
18	R 30	1	106′ 0″	6	17′ 8″	12′ 0″	Two 32-in.	22-in.	Two 10-in. ⊔'s	143 300	1896	Pony truss.
19	R 14	1	126′ 0″	6	21′ 0″	39½-in.	28¼-in.	{ Four 5 x 3½-in. L's. / One 8½-in. pl }	176 000	1896	Lower chord stiff.
20	Columbus Bridge.	1	200′ 0″	7	28′ 7″	{ 26′ 6″ / 24′ 0″ / to / 38′ 8″ }	42-in. (suspended)	45-in.	.....	265 000±	1893	Pegram truss. East span of bridge.
21	Columbus Bridge.	1	147′ 6″	6	24′ 7″	{ 23′ 0″ / to / 30′ 0″ }	42-in. (suspended)	45-in.	.....	165 000±	1893	Pegram truss. Second span from east end.
22	Elkhorn R. Bridge.	8	147′ 1½″	11	13′ 4½″	21′ 0″	20-in. (suspended)	.....	Two 4 x ⅞-in.	175 000±	Old spans	Tested east one of old spans — third span of bridge.
23	Elkhorn R. Bridge.	1	146′ 6⅜″	9	16′ 3⅜″	24′ 0″	30-in. (suspended)	I beams.	Two 3 x 1½-in.	175 000±	.....	Trestle approach at east end.

TABLE No. 3.—Results of Tests on Plate Girders.

Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	Deflection, in inches.	Percentage added to deflection by vibration.	Flange stress, in pounds per square inch.	Percentage added to flange strains by vibration.	+ Vibrations per second.	Revolutions of drivers per second.	Remarks.
No. 1.—95-ft. span.	49	F. E.	824*	.....	....	20	.....	....	7 000	28	4	.....	Vibrations irregular.
	46	F. E.	659*	189 100	62	36	0.12	....	4 000	25	4	3.4	
	45	P. W.	431	.....	....	45	.....	....	5 700	47	8	.....	Excessive instrumental vibrations.
	52	P. W.	.....	.....	....	46	.....	....	5 200	35	5	.....	
	48	P. E.	311	154 000	62	50	.....	....	4 300	25	5	4.6	
	47	P. E.	725	178 200	68	50	.....	....	5 200	42	5	4.2	
No. 2.—95-ft. span.	234	F. W.	480	146 150	62	24	0.125	16	4 500	26	Irreg.	2.2	
	261	F. E.	358	141 000	69	25	0.10	20	3 800	47?	4	2.1	
	260	P. W.	538	141 000	69	27	0.11	15	4 800	35?	4	2.2	
	232	F. W.	496	146 150	62	30	0.12	25	4 700	30	3	2.8	
	237	F. W.	487	146 150	62	30	0.13	25	4 300	?	.....	2.8	
	233	P. W.	546	141 000	69	35	0.11	35	4 500	?	.....	2.9	
	238	F. E.	442	146 150	62	37	.....	....	4 400	50?	.....	3.4	
	235	P. E.	367	141 000	69	50	0.10	40	5 000	75?	5	4.1	
	236	P. W.	541	141 000	69	....	0.10	30	3 800	26	5	.....	
No. 3.—90-ft. span.	162	F. E.	439	146 150	62	16	0.12	0	4 000	15	2	1.5	
	157	F. E.	442	146 150	62	23	.....	....	3 700	10	2.5	2.2	
	160	F. W.	358	141 000	69	26	.....	....	3 700	13	.....	2.2	
	163	F. W.	480	146 150	62	33	.....	....	4 000	30	3	3.0	
	159	P. W.	546	141 000	69	41	.....	....	3 800	34?	4	3.4	
	164	P. E.	360	141 000	69	41	.....	....	4 300	46	4	3.4	
	158	P. E.	367	141 000	69	43	.....	....	3 800	35	4	3.6	
	161	P. W.	541	141 000	69	47	.....	....	3 800	40	4	3.9	
No. 4.—85-ft. span.	223	F. W.	.....	.....	....	.....	.....	....	3 500	17	5	.....	Ten other experiments with speeds from 4 to 24 miles per hour gave no increase in stress due to vibration greater than 10 per cent.
	220	F. W.	553	146 150	62	25	0.15	16	3 800	18	5	2.3	
	224	P. W.	545	141 000	69	27	.....	....	3 300	30	4	2.2	
	213	P. W.	367	141 000	69	29	.....	....	4 000	20	3	2.4	
	214	P. E.	365	141 000	69	35	0.13	20	4 000	30	3	2.9	
	211	F. E.	486	146 150	62	35	0.15	16	4 000	25	3	3.2	
	224½	P. E.	541	141 000	69	37	.....	....	3 400	18	3.5	3.1	
	218	P. W.	365	141 000	69	41	0.15	25	3 000	50?	4	3.4	
	216	P. W.	362	141 000	69	43	0.14	21	3 800	40?	4	3.6	
No. 5.—35-ft. span.	62	F. E.	824*	.....	....	9	0.20	0	5 800	0	.....	.....	
	42	F. W.	.....	.....	....	20	.....	....	4 200	7	.....	.....	
	60	F. W.	715*	189 100	62	25	.....	....	3 700	0	.....	2.3	
	61	F. W.	639*	.....	....	28	.....	....	3 200	12	Irreg.	Irreg.	
	58	F. E.	648*	.....	....	32	0.18	11	4 800	21	3.5	.....	
	43	P. W.	311	154 000	62	33	.....	....	4 200	60?	3.5	3.1	
	44	P. W.	722	178 200	68	33	.....	....	4 000	26?	7	2.8	Instrumental vibration.
	55	F. E.	701*	189 100	62	39	0.17	45	4 500	49	4	3.6	
	53	P. W.	.....	.....	....	40	.....	....	3 300	30	4	.....	
	57	P. E.	340	.....	....	50	.....	....	4 000	40?	5	.....	
	56	P. E.	732	178 200	68	60	.....	....	4 200	95?	6	5.0	
No. 6.—45-ft. span.	63	F. W.	672*	189 100	62	19	0.23	6	4 200	10	4	1.7	
	"	"	"	Cars.	.....	.....	.....	0.10	15	1 600	15	3.3	.....
	66	F. W.	761*	189 100	62	20	.....	....	4 400	9	.....	1.8	
	"	"	"	Cars.	.....	.....	.....	.....	....	2 400	15	3.5	.....
	70	F. W.	643*	.....	....	21	.....	....	4 100	15	Irreg.	.....	
	72	F. E.	709*	189 100	62	27	0.29	34	5 700	33	3	2.5	
	71	F. E.	643*	.....	....	29	.....	....	4 500	31	3	.....	
	74	F. E.	643*	.....	....	29	0.27	13	5 000	12	3	.....	
	75	F. E.	695*	189 100	62	33	0.24	42	5 000	65	3.3	3.1	
	64	P. W.	431	.....	....	33	0.17	6	3 300	18	6	.....	
	69	F. E.	739	178 200	68	38	.....	....	4 500	44	4	3.2	
	65	P. E.	311	154 000	62	49	0.25	20	4 000	19	5	4.5	
	67	P. E.	725	178 200	68	50	0.29	24	4 700	42?	5	4.2	

* Ten-wheel engines; all others of American type.

† Roughly approximate when given in whole numbers.

TABLE No. 3—*(Continued).*

Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	Deflection, in inches.	Percentage added to deflection by vibration.	Flange stress, in pounds per square inch.	Percentage added to flange strains by vibration.	† Vibrations per second.	Revolutions of drivers per second.	Remarks.
No. 7.—45-ft. span.	139	F. E.	83	.....	....	23	0.15	0	2 600	0	.....	.....	
	142	F. W.	83	.....	....	24	0.17	0	2 800	0	.....	.....	
	143	F. E.	504	146 150	62	24	0.20	0	2 700	9	5	2.2	
	141	F. W.	480	146 150	62	24	0.22	?	4 000	16	Irreg.	2.2	
	135	F. E.	442	146 150	62	27	0.19	19	3 300	15	3.0	2.5	
	138	F. W.	496	146 150	62	28	0.21	12	3 700	11	3.0	2.6	
	137	P. W.	546	141 000	69	35	.....	....	3 000	20	3.7	2.9	
	134	P. E.	360	141 000	69	35	.....	....	3 300	30	?	2.9	
	136	P. E.	538	141 000	69	37	.....	....	4 000	19	3.5	3.1	
	144	P. E.	362	141 000	69	41	0.20	20	3 500	43	4	3.4	
	140	P. W.	541	141 000	69	59	.....	....	3 200	50?	?	4.9	
No. 8.—55-ft. span.	2	F. W.	761*	189 100	62	37	.....	....	4 700	15	3.7	3.4	
	3	F. W.	773*	189 100	62	37	.....	....	4 750	21	3.7	3.4	
	5	F. W.	720*	189 100	62	44	0.30	12	4 000	20	4.6	4.1	
	6	F. W.	717*	189 100	62	44	0.31	20	4 500	22	4.3	4.1	
	4	F. E.	486	146 150	62	50	0.26	32	4 300	47	4.7	4.6	
	7	P. W.	731	178 200	68	54	0.26	25	3 900	31	5.0	4.5	
No. 9.—60-ft. span.	168	F. W.	496	146 150	62	24	0.22	11	3 800	16	3	2.2	Two other tests at 24 miles per hour gave 5% increase due to vibration.
	170	F. W.	442	146 150	62	30	0.25	20	4 100	18	3	2.8	
	172	F. W.	504	146 150	62	30	0.22	11	3 350	10	3.5	2.8	
	171	P. E.	361	141 000	69	32	0.22	10	3 200	16	3.2	2.7	
	166	P. E.	545	141 000	69	34	0.22	17	3 750	21	3.7	2.8	
	167	P. W.	360	141 000	69	44	0.22	12	3 800	16	4.3	3.6	
	169	P. W.	362	141 000	69	44	0.24	19	3 900	22	4	3.6	
No. 10.—70-ft. span.	83	F. E.	777*	189 100	62	35	.....	....	2 200	18	4	3.3	Five other tests at speeds from 12 to 35 miles per hour gave no increase due to vibration greater than 8 per cent.
	81	P. E.	340	154 000	62	37	0.17	0	1 750	8	3.5	3.4	
	84	F. W.	781*	189 100	62	40	.....	....	1 800	14	4.5	3.7	
	"	" "	Cars.	.....	....	....	.....	....	1 000	20	3	.....	
	86	F. W.	767*	189 100	62	40	0.17	14	1 800	14	4	3.7	
	"	" "	Cars.	.....	....	....	0.095	16	900	22	2.8	.....	
	87	F. W.	715*	189 100	62	43	0.16	19	1 700	28	4	4.0	
	88	F. W.	668*	189 100	62	43	0.16	25	1 700	23	4	4.0	
	76	P. W.	.....	.....	....	47	0.12	17	1 200	21	4.5	.....	
	80	P. E.	.....	.....	....	54	0.18	18	1 700	23	5	.....	
No. 11.—52-ft. skew span.	316	E.†	454‡	170 600	56	12	0.27	0	4 000	5	.....	1.2	
	315	W.	.....	.....	....	13	0.25	0	3 900	5	.....	1.3	
	317	W.	.....	.....	....	20	0.27	0	4 000	7	.....	2.0	
	319	W.	.....	.....	....	28	0.27	9	4 000	12	3.0	2.9	
	318	E.	.....	.....	....	29	0.27	?	4 100	12	3.0	3.0	
	320	E.	.....	.....	....	40	0.27	20	.....	.....	4.3	4.1	
	321	W.	.....	.....	....	43?	0.26	15	4 000	15	4.6	4.4	
	322	E.	.....	.....	....	43	0.27	18	4 000	21	4.6	4.4	
No. 12.—85-ft. span.	309	W.	.....	.....	....	20	0.35	3	2 700	3	4	2.0	Four other experiments at speeds from 6 to 17 miles per hour gave no appreciable vibration.
	"	"	Cars.	.....	....	....	0.22	11	1 600	9	3	.....	
	311	W.	.....	.....	....	23	0.35	6	2 700	Small	5	2.3	
	313	W.	.....	.....	....	31	0.35	9	2 750	6	3.5	3.2	
	310	E.	.....	.....	....	35	0.35	10	2 700	9	4.0	3.6	
	312	E.	.....	.....	....	41	0.35	17	2 700	15	4.4	4.2	
	314	E.	.....	.....	....	53	0.34	14	2 700	13	6	5.4	
	"	"	Cars.	.....	....	....	0.22	14	1 600	16	.....	.....	

* Ten-wheel engines.

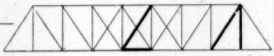
† All experiments on bridges Nos. 11 and 12 were made with a special train which always faced toward the east.

‡ Engine 454 is a ten-wheel engine.

TABLE No. 4.—RESULTS OF TESTS ON TRUSSES.

Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	DEFLECTION. Mean deflection, in inches.	DEFLECTION. Percentage added by vibration.	STRESS IN CHORD. Mean stress, in pounds per square inch.	STRESS IN CHORD. Percentage added by vibration.	STRESS IN DIAGONAL Mean stress, in pounds per square inch.	STRESS IN DIAGONAL Percentage added by vibration.	STRESS IN HIP. Mean stress, in pounds per square inch.	STRESS IN HIP. Percentage added by vibration.	Vibrations per second.	Revolutions of drivers per second.
No. 13.—158-ft. span.	8	F. W.	714	189 100	62	27	....	....	4 200	24	......	....	......	....	2.5	2.5
	26	F. W.	777	189 100	62	27	0.68	13	......	....	6 700	13	......	....	2.7	2.5
	"	" "	Cars.	....	....	....	0.42	20	......	....	3 300	27	......	....	2.4	....
	28	F. W.	747	189 100	62	27	0.65	17	......	....	6 300	25	......	....	2.4	2.5
	"	" "	Cars.	....	....	....	0.44	26	......	....	3 500	40	......	....	2.2	....
	37†	F. W.	709	189 100	62	27	0.70	20	......	....	6 800	21	......	....	2.5	2.5
	10	F. E.	773	189 100	62	28	0.75	19	4 300	19	......	....	......	....	2.8	2.6
	"	" "	Cars.	....	....	....	0.58	26	2 800	25	......	....	......	....	2.5	....
	14	F. E.	720	189 100	62	28	0.72	14	3 700	11	......	....	......	....	2.8	2.6
	"	" "	Cars.	....	..	....	0.50	30	2 700	33	......	....	......	....	2.8	....
	23	F. W.	788	189 100	62	28	0.71	11	4 000	10	......	....	......	....	3.2	2.6
	31	F. E.	......	....	....	28	0.71	17	......	....	......	....	......	....	2.4	....
	"	" "	Cars.	....	....	....	0.57	23	......	....	......	....	......	....	2.0	....
	12	F. W.	793	189 100	62	29	0.67	6	3 700	8	......	....	......	....	2.8	2.7
	25	F. W.	720	189 100	62	30	....	....	3 800	13	......	....	......	....	....	2.8
	38†	F. E.	747	189 100	62	30	0.69	12	......	....	2 000	55	......	....	3.0	2.8
	11	F. E.	788	189 100	62	31	0.80	6	......	....	......	....	......	....	....	2.9
	"	" "	Cars.	....	....	....	0.45	18	......	....	......	....	......	....	....	....
	32	F. E.	788	189 100	62	31	0.72	10	......	....	5 200	20	......	....	3	2.9
	33*	F. W.	781	189 100	62	31	0.52 0.67	17 5	....	....	6 100	16	......	....	3	2.9
	27*	F. E.	794	189 100	62	32	0.57 0.73	14 10	....	....	5 400	15	......	....	3	3.0
	29	P. E.	721	178 200	68	33	0.64	9	......	....	5 000	10	......	....	3.3	2.8
	35	F. E.	720	189 100	62	33	0.71	5	......	....	5 700	11	......	....	3	3.1
	"	" "	Cars.	....	....	....	0.51	16	......	....	4 300	21	......	....	2.5	....
	9	P. E.	734	178 200	68	37	0.80	5	4 400	7	......	....	......	....	3.7	3.1
	34	P. W.	731	178 200	68	37	0.66	8	......	....	4 400	14	......	....	4	3.1
	40†	P. E.	......	....	....	37	....	....	......	....	5 300	13	......	....	3.3	....
	13	P. W.	731	178 200	68	40	0.67	4	4 500	5	......	....	......	....	3.5	3.4
	22	F. W.	761	189 100	62	40	0.70	11	4 000	10	......	....	......	....	3.5	3.7
	24	P. W.	732	178 200	68	40	0.69	5	4 100	10	......	....	......	....	3.4	3.4
	39†	P. E.	721	178 200	68	40	0.67	5	......	....	4 100	22	......	....	4	3.4
	36	P. W.	......	....	....	47	0.68	5	......	...	6 200	11	......	....	4.5	....
	41	P. W.	722	178 200	68	50	....	....	......	....	5 800	24	3 200	72?	5	4.2

East ←

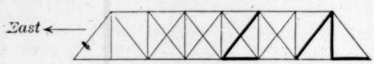


All freight engines are ten-wheelers.

* Maximum vibration occurs before maximum deflection in Nos. 33 and 27.

† Tests Nos. 37, 38, 40 and 39 were made on center diagonal, all others on end diagonal.

TABLE No. 4—(*Continued*).



Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	Deflection. Mean deflection, in inches.	Deflection. Percentage added by vibration.	Stress in Chord. Mean stress, in pounds per square inch.	Stress in Chord. Percentage added by vibration.	Stress in Diagonal. Mean stress, in pounds per square inch.	Stress in Diagonal. Percentage added by vibration.	Stress in Hip. Mean stress, in pounds per square inch.	Stress in Hip. Percentage added by vibration.	Vibrations per second.	Revolutions of drivers per second.
No, 14.—175-ft. span.	93	F. E.	......	....	....	16	0.41	5	2 500	4	......	....	......	....	2.6	....
	110	F. E.	426	126 600	57	20	0.46	2	......	....	3 000	7	......	....	3.3	....
	97	F. W.	403	121 000	57	23	0.41	0	2 200	0	......	....	2 800	5	....	....
	92	L. W.	637*	182 000	56	25	0.39	5	2 400	6	......	....	......	....	2.5	....
	94	P. E.	411	119 150	56	25	0.37	3	2 250	4	......	....	......	....	3	....
	99	F. W.	403	121 000	57	25	0.41	2	2 300	0	......	....	2 800	2	3.5	....
	101	F. W.	403	121 000	57	28	0.41	6	2 200	4	......	....	2 800	9	3.0	....
	90	F. W.	403	121 000	57	31	0.36	5	2 200	4	......	....	......	....	....	....
	95†	P. W.	407	121 000	56	31	0.35	3	1 800	0	......	....	......	....	....	....
	105‡	F. W.	403	121 000	57	34	0.41	2	......	..	3 400	4.	end..	....	3	....
											2 500	14.	c'nt'r	....	7	....
	103‡	F. W.	403	121 000	57	36	0.41	10	......	..	3 400	5.	end..	....	3	....
											2 500	16.	c'nt'r	....	7	....
	107‡	F. W.	403	121 000	57	40	0.41	5	......	..	3 400	3.	end..	....	....	....
											2 000	0.	c'nt'r	....	....	....
	96	P. E.	411	119 150	56	44	0.33	7	2 500	16	......	....	2 500	16	4	....
	91	P. W.	411	119 150	56	47	0.33	12	1 800	8	......	....	......	....	4.4	....

† Test No. 95 was made on end chord, all others on chord near center.

‡ Vibrations recorded for center diagonal were due to vibration of bar and not of bridge.

Seven unrecorded tests at speeds from 8 to 15 miles per hour showed no appreciable vibrations.

No. 15.—104-ft. span.	125	F. E.	480	146 150	62	23	0.41	0	5 700	5	......	....	......	....	....	2.1
	"	" "	Cars.	....	....	....	0.16	12	1 300	11	......	....	......	....	4.5	....
	130	F. W.	442	146 150	62	24	0.37	5	......	....	3 300	18	......	....	....	2.2
	118	F. W.	480	146 150	62	25	0.36	7	1 800	9	......	....	......	....	2.5	2.3
	114	F. E.	442	146 150	62	26	0.36	5	5 800	9	......	....	......	....	....	2.4
	"	" "	Cars.	....	....	....	0.26	10	3 000	18	......	....	......	....	....	....
	146	F. E.	.....	....	....	27	0.39	6	......	....	5 500	5	6 800	9	3	....
	132	F. W.	439	146 150	62	28	0.38	13	6 500	17	2 100	38	......	....	3	2.6
	"	" "	Cars.	....	....	....	0.30	28	5 000	24	2 100	60	......	....	3	....
	148	F. W.	553	146 150	62	28	0.35	5	......	....	5 200	8	8 400	19?	3	2.6
	154	F. E.	487	146 150	62	29	0.35	6	......	....	......	....	......	....	3	2.7
	126	F. W.	504	146 150	62	30	0.39	12	4 300	12	......	....	......	....	3.1	2.8
	152	F. W.	442	146 150	62	31	0.36	13	......	....	......	....	8 500	16?	3.6	2.9
	149	F. E.	358	141 000	69	33	0.38	9	......	....	......	....	7 500	12	3	2.7
	"	" "	Cars.	....	....	....	0.23	17	......	....	......	....	......	....	2.5	....
	155	P. E.	546	141 000	69	33	0.37	12	......	....	......	....	5 000	20	3	2.7
	133	P. E.	546	141 000	69	34	....	....	......	....	8 500	8	......	....	4	2.8
	150	P. W.	365	141 000	69	37	0.35	14	......	....	......	....	......	....	....	3.1
	127	P. W.	362	141 000	69	37	0.37	10	4 300	12	......	....	......	....	4	3.1
	121	F. E.	496	146 150	62	39	....	....	4 400	23	......	....	......	....	....	3.6
	"	" "	Cars.	....	....	....	....	....	3 000	20	......	....	......	....	....	....
	115	P. W.	541	141 000	69	39	0.37	6	......	....	......	....	......	....	....	3.2

Six other tests at speeds from 7 to 18 miles per hour gave no appreciable vibration.

* Ten-wheel engine.

TABLE No. 4—(*Continued*).

Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	Deflection. Mean deflection, in inches.	Deflection. Percentage added by vibration.	Stress in Chord. Mean stress, in pounds per square inch.	Stress in Chord. Percentage added by vibration.	Stress in Diagonal. Mean stress, in pounds per square inch.	Stress in Diagonal. Percentage added by vibration.	Stress in Hip. Mean stress, in pounds per square inch.	Stress in Hip. Percentage added by vibration.	Vibrations per second.	Revolutions of drivers per second.
No. 15, continued.	147*	P. W.	362	141 000	69	41	0.37	13	.....	....	4 500	33	7 000	45	....	3.4
	113	P. E.	538	141 000	69	42	0.36	14	4 800	13	.....	....	.....	....	4	3.5
	120	P. E.	362	141 000	69	42	0.36	20	7 000	19	.....	....	.....	....	3.8	3.5
	122	F. W.	10	.....	....	43	.....	....	3 750	8	.....	....	.....	....	3.5	....
	123	F. E.	10	.....	....	43	0.29	7	3 800	5	.....	....	.....	....	3.6	....
	117	P. W.	367	141 000	69	44	0.35	10	.....	....	.....	....	.....	....	....	3.6
	129	P. W.	365	141 000	69	44	0.39	7	.....	....	4 800	12	.....	....	4	3.6
	145	P. E.	361	141 000	69	45	0.27 0.38	19 10	.....	....	4 800	30	6 000	30?	4	3.7
	124	P. E.	361	141 000	69	47	0.29	12	4 000	12	.....	....	.....	....	4	3.9

* Increase in stress in diagonal and hip due chiefly to vibration of the individual bars.

No. 16—127 ft. span.	178	F. E.	442	146 150	62	20	0.42	7	2 800	7	.....	....	.....	....	4	1.8
	202	F. E.	492	146 150	62	24	0.43	6	.....	....	4 800	6	2 300	15	3.3	2.2
	188	F. W.	584	146 150	62	25	0.35	8	3 200	6	.....	....	.....	....	2.7	2.3
	181	P. W.	360	141 000	69	28	0.36	7	2 900	10?	.....	....	.....	....	3.6	2.3
	184	F. E.	486	146 150	62	28	0.37	6	2 800	5	4 800	6	.....	....	2.9	2.6
	"	"	Cars.	.....	....	....	0.14	21	800	25	500	80	.....	....	4.7	....
	187	F. W.	439	146 150	62	28	0.34	12	3 000	5	.....	....	.....	....	3	2.6
	201	P. E.	367	141 000	69	31	0.38	21	.....	....	4 600	22	2 500	40?	3.2	2.6
	195	F. W.	358	141 000	69	31	0.36	11	.....	....	.....	....	4 000	12	3.2	2.6
	190	F. E.	496	146 150	62	31	0.42	18	.....	....	5 200	21	.....	....	3.2	2.9
	193	F. E.	504	146 150	62	32	0.42	15	.....	....	.....	....	3 500	20	3.1	3.0
	179	P. E.	365	141 000	69	36	0.40	12	2 800	11	.....	....	.....	....	....	3.0
	200	F. E.	439	146 150	62	36	0.44	11	.....	....	4 800	25	2 500	28	3.2	3.3
	"	"	Cars.	.....	....	....	0.31	15	.....	....	1 900	58	.....	....	2.7	....
	196	F. E.	480	146 150	62	37	0.43	4	.....	....	.....	....	3 500	14	3.4	3.4
	197	P. E.	541	141 000	69	37	0.41	7	.....	....	4 600	15	2 300	11	3.7	3.1
	185	P. E.	361	141 000	69	37	0.39	15	2 900	14	4 800	23	.....	....	3.5	3.1
	191	P. E.	546	141 000	69	40	0.37	11	.....	....	4 300	23	.....	....	3.7	3.3
	177	F. W.	496	146 150	62	40	0.39	11	2 900	17	.....	....	.....	....	4	3.7
	174	P. E.	362	141 000	69	41	0.40	18	3 000	16	.....	....	.....	....	3.7	3.4
	199	P. E.	538	141 000	69	43	0.38	13	.....	....	4 600	19	2 100	19	4.1	3.6
	176	P. W.	367	141 000	69	45	0.37	11	2 500	12	.....	....	.....	....	4	3.7
	186	P. W.	362	141 000	69	45	0.37	17	2 800	16	3 900	28	.....	....	3.8	3.7
	182	P. E.	545	141 000	69	47	0.36	20	2 900	17	4 500	18	.....	....	4	3.9
	194	P. W.	545	141 000	69	50	0.36	14	.....	....	.....	....	3 000	35	4.2	4.1

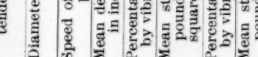
Eight other tests at speeds from 8 to 26 miles per hour showed no appreciable vibration.

TABLE No. 4—(*Continued*).

Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	Deflection. Mean deflection, in inches.	Deflection. Percentage added by vibration.	Stress in Chord. Mean stress, in pounds per square inch.	Stress in Chord. Percentage added by vibration.	Stress in Diagonal. Mean stress, in pounds per square inch.	Stress in Diagonal. Percentage added by vibration.	Stress in Hip. Mean stress, in pounds per square inch.	Stress in Hip. Percentage added by vibration.	Vibrations per second.	Revolutions of drivers per second.
									East ←							
No. 17.—123-ft. span.	259	P. E.	538	141 000	69	27	0.28	5	.....	....	.....	....	2 400	17	4	2.2
	228	P. E.	546	141 000	69	27	....	....	.....	....	.....	....	2 750	7	4	2.2
	226	F. W.	353	.....	....	29	....	....	.....	....	.....	....	3 400	26	....	....
	241	P. E	362	141 000	69	30	....	....	2 700	8	.....	....	.....	....	3	2.5
	257	F. E.	504	146 150	62	31	0.28	12	.....	....	.....	....	2 700	11	2.9	2.9
	251	F. E.	553	146 150	62	32	0.27	6	.....	....	1 300	0	.....	....	3.8	3.0
	"	" "	Cars.	.....	....	....	0.30	12	2 700	11	.....	....	.....	....	3.6	....
	229	F.	487	146 150	62	....	....	....	.....	....	.....	....	3 000	26?	....	....
	240	P. W.	538	141 000	69	33	....	....	2 000	15	.....	....	.....	....	4	2.7
	252	P. W.	361	141 000	69	33	0.26	19	.....	....	2 800	12	.....	....	4	2.7
	250	P. W.	365	141 000	69	38	0.27	13	.....	....	2 000	20	.....	....	3.5	3.1
	246	P. W.	362	141 000	69	39	0.30	20	2 500	16	2 800	21	.....	....	3.6	3.2
	254	P. W.	546	141 000	69	40	0.25	16	.....	....	2 900	14	2 600	27	3.6	3.4
	243	P. W.	360	141 000	69	43	....	....	2 700	15	.....	....	.....	....	3.8	3.5
	225	P. W.	365	141 000	69	45	....	....	.....	....	.....	....	2 800	36	4.5	3.7
	230	P. W.	545	141 000	69	47±	....	....	.....	....	.....	....	2 600	31	5	3.9
	231	P. E.	.....	.....	....	47±	....	....	.....	....	.....	....	2 700	26	....	....
	Eight other tests at speeds from 16 to 25 miles per hour showed very small vibration.															
									East ←							
No. 18—106-ft. span.	299	W.*	454*	170 600	56	30	0.50	8	.....	....	.....	....	1 250	0	3.8	2.0
	292	E.	.....	.....	....	26	0.50	7	3 900	8	3 200	0	.....	....	3.0	2.6
	295	W.	.....	.....	....	27	0.50	7	3 900	10	3 400	12	.....	....	3.5	2.7
	293	W.	.....	.....	....	28	0.50	7	3 900	5	3 300	0	.....	....	3.4	2.8
	301	W.	.....	.....	....	29	0.48	4	.....	....	.....	....	.....	....	3.6	3.0
	303	W.	.....	.....	....	32	0.47	12	.....	....	.....	....	.....	....	3.7	3.3
	300	E.	.....	.....	....	35	0.48	23	.....	....	.....	....	1 300	15	3.7	3.6
	294	E.	.....	.....	....	40	0.49	15	3 800	11	3 400	5	.....	....	4	4.1
	302	E.	.....	.....	....	45	0.50	5	.....	....	.....	....	.....	....	4.4	4.6
	"	"	Cars	.....	....	....	0.36	35	.....	....	.....	....	.....	....	3.0	....
	296	E.	.....	.....	....	47	0.50	5	3 900	5	3 400	9	.....	....	3.6	4.8
	"	"	Cars.	.....	....	....	0.37	35	2 800	32	1 200	46	.....	....	2.7	....
	304	E.	.....	.....	....	50	0.50	5	.....	....	.....	....	.....	....	....	5.1
	"	"	Cars.	.....	....	....	0.36	35	.....	....	.....	....	600	42	3.1	....
	Five other tests at speeds from 10 to 16 miles per hour gave no appreciable vibration.															
									East ←							
No. 19.—126-ft. span.	276	E.	.....	.....	....	27	0.33	3	1 800	0	3 600	6	.....	....	3.5	2.8
	284	E.	.....	.....	....	28	0.34	6	.....	....	.....	....	3 850	17	3.7	2.9
	286	E.	.....	.....	....	31	0.32	12	.....	....	.....	....	4 000	15	3.7	3.2
	278	E.	.....	.....	....	35	0.33	10	1 800	11	3 500	9	.....	....	3.9	3.6
	288	E.	.....	.....	....	39	0.32	7	.....	....	.....	....	3 800	26	4	4.0
	"	"	Cars.	.....	....	....	0.25	20	.....	....	.....	....	.....	....	....	....
	280	E.	.....	.....	....	50	0.33	13	.....	....	.....	....	.....	....	....	5.1
	Ten other tests at speeds from 10 to 26 miles per hour gave no appreciable vibration.															

* All experiments on bridges Nos. 18 and 19 were made with a special train which always faced toward the east. Engine No. 454 is a ten-wheel engine.

TABLE No. 4—(*Continued*).

Bridge No.	No. of experiment.	Kind and direction of train.	No. of engine.	Weight of engine and tender, in pounds.	Diameter of drivers, in inches.	Speed of train, in miles per hour.	Deflection. Mean deflection, in inches.	Deflection. Percentage added by vibration.	Stress in Chord. Mean stress, in pounds per square inch.	Stress in Chord. Percentage added by vibration.	Stress in Diagonal. Mean stress, in pounds per square inch.	Stress in Diagonal. Percentage added by vibration.	Stress in Hip. Mean stress, in pounds per square inch.	Stress in Hip. Percentage added by vibration.	Vibrations per second.	Revolutions of drivers per second.
Seven-panel Pegram truss. Center chord member tested.																
No. 20.—200-ft. span.	325	F. E.	774	169 500	62	20	0.41	6	2 200	9	.....	....	.....	....	....	1.9
	323	F. E.	799	169 500	62	....	....	....	2 900	14	.....	....	.....	....	3	....
	324	P. W.	.....	.....	....	24	0.57	5	2 750	18	.....	....	.....	....	2.5	....
	332	P. W.	890	226 800	69	28	....	....	3 200	11	.....	....	.....	....	2.5	2.3
	326	P. E.	1 800*	216 500	69	30	....	....	1 400	14	.....	....	.....	....	2.7	2.5
	331	P. E.	1 801*	216 500	69	44	0.32	12	3 300	10	.....	....	.....	....	2.5	3.6
* Engines 1 800 and 1 801 are ten-wheelers.																
East ← 																
No. 21.—147-ft. span.	334	F. W.	779	169 500	62	11	....	....	2 750	5	1 600	10	.....	....	2.4	1.0
	343	F. W.	771	169 500	62	22	....	....	3 000	5	4 000	11	.....	....	2.7	2.0
	339	F. W.	773	169 500	62	23	....	....	2 400	10	3 800	13	.....	....	2.7	2.1
	345	F. E.	799	169 500	62	25	0.51	10	4 000	10	4 100	12	.....	....	2.7	2.3
	335	P. W.	845	213 000	69	25	....	....	2 800	7	4 800	6	.....	....	2.5	2.1
	338	F. E.	670	169 500	62	25	....	....	2 800	7	4 200	12	.....	....	2.5	2.3
	347	F. W.	700	169 500	62	25	....	....	2 800	7	3 300	6	.....	....	2.5	2.3
	348	F. W.	722	169 500	62	27	....	....	2 800	14	3 300	12	.....	....	3	2.5
	340	P. W.	771	169 500	62	30	....	....	3 000	0	3 600	11	.....	....	2.6	2.8
	341	P. W.	1 801	216 500	69	32	....	....	3 500	17	4 000	12	.....	....	2.8	2.6
	337	P. E.	890	226 800	69	40	....	....	2 600	12	4 900	10	.....	....	3	3.3
Six other tests at speeds from 10 to 18 miles per hour gave no vibration.																
East ←																
No. 22.—147-ft. span.	362	F. E.	641	169 500	62	20	0.85	16	5 800	17	2 500	18	.....	....	2.1	1.9
	355	F. E.	792	169 500	62	21	0.81	16	5 800	15	2 500	18	.....	....	2.6	2.0
	365	F. E.	700	169 500	62	23	....	....	5 800	10	2 800	9	.....	....	2.3	2.1
	366	F. W.	773	169 500	62	24	....	....	5 800	4	3 700	5	.....	....	2.2	2.2
	369	F. W.	799	169 500	62	25	....	....	5 700	15	3 800	8	.....	....	2.4	2.3
	357	P. E.	1 801	216 500	69	27	....	....	6 800	7	3 300	12	.....	....	2.2	2.2
	364	F. E.	722	169 500	62	27	....	....	5 200	11	2 800	4	.....	....	2.4	2.5
	368	F. E.	.....	.....	....	27	....	....	4 500	18	2 100	17	.....	....	2.2	....
	367	F. E.	643	169 500	62	31	....	....	6 500	3	2 800	0	.....	....	....	2.9
	356	P. E.	777	169 500	62	40	....	....	6 100	18	2 800	?	.....	....	3.0	3.7
Two other tests at a speed of 15 miles per hour gave 5% increase due to vibration.																
East ←																
No. 23.—146-ft. span.	375	F. W.	722	169 500	62	23	0.57	11	.....	....	3 300	24	.....	....	2.8	2.1
	380	F. E.	671	169 500	62	25	....	....	.....	....	3 500	14	3 300	0	....	2.3
	372	P. E.	845	213 000	69	35	0.67	7	.....	....	4 200	7	.....	....	....	2.9
	379	F. W.	672	169 500	62	37	....	....	.....	....	4 200	26	3 300	19	4	3.4
	376	P. E.	490	136 000	68	40	....	....	.....	....	4 200	14	3 300	33	3.6	3.4
	371	P. W.	844	213 000	69	46	0.63	10	.....	....	4 200	5	.....	....	....	3.8
	378	P. W.	1 800	216 500	69	50	....	....	.....	....	5 800	8	5 000	10	....	4.1
	381	P. W.	.....	.....	....	50	....	....	.....	....	.....	....	5 200	20†	....	....
	377	P. W.	775	169 500	62	52	....	....	.....	....	4 800	25	3 600	47	4	4.8
Two other tests at speeds of 14 and 18 miles per hour gave no appreciable vibration.																

†Due largely to vibration of bar.

TABLE No. 5.—Summary of Tests on Trusses.

Bridge No.	Span length.	Deflection under heavy train.	Ratio of deflection to span × 100 000.	Panel lengths.	Speeds causing over 10 per cent. vibration from the locomotive. Miles per hour.	Maximum percentage of increase in deflection due to vibration from the locomotive.	Rate of driver-revolution for speeds causing over 10 per cent. vibration. Number per second.	Rate of bridge vibration when vibration is over 10 per cent. Number per second.	Calculated rate of vibration of loaded bridge. Number per second.	Maximum percentage of increase in deflection due to vibration from cars.
13.....	158′ 3″	0.72″	38	17′ 9″	— 27 to 40	20	2.5 to 3.7	2.5 to 4	3.6	30
14.....	175′ 0″	0.41″	19	21′ 10½″	36 " 47 +	12	.....	3 " 4.4	4.2	8
15.....	104′ 0″	0.39″	31	17′ 4″	22 " 47 +	20	2.6 to 3.9	3 " 4	5.0	28
16.....	127′ 9″	0.42″	27	18′ 3″	28 " 50 +	21	2.6 " 4.1	3 " 4	4.5	21
17.....	123′ 6″	0.28″	19	20′ 7″	21 " 40 +	20	2.9 " 3.4	3 " 4 +	5.5	12
18.....	106′ 0″	0.50″	39	17′ 8″	32 " 40	13	3.3 " 4.1	3.6 " 4 +	4.2	35
19.....	126′ 0″	0.33″	22	21′ 0″	31 " 50 +	13	3.2 " 5.1	3.7 " 4 +	5.0	20
20.....	200′ 0″	0.5″*	21	28′ 7″	— 24 " 44 +	18	1.9 " 3.6	2.5 " 3	4.2	Small
21.....	147′ 6″	0.5″*	28	24′ 7″	23 " 40 +	17	2.1 " 3.3	2.5 " 3	4.3	"
22.....	147′ 1½″	0.8″*	45	13′ 4½″	20 " 40 +	28	1.9 " 3.7	2.1 " 3	3.3	7
23.....	146′ 6⅜″	0.6″*	34	16′ 3⅜″	23 " 52 +	26†	2.1 " 4.8	2.8 " 4	4.0	16

* Not well determined.

† Diagonal.

DISCUSSION.

H. B. SEAMAN, M. Am. Soc. C. E.—The method of making the experiments described by the author is open to the criticism that two results are compared which are not directly comparable. The records obtained by the instrument would appear to represent the actual conditions of strain, as nearly as can be ascertained in practice, but the calculations of static strains are much less reliable, as they include many indeterminate conditions, and should be replaced by actual strain diagrams. For this purpose it is necessary, in making the diagram of dynamic strain, to have in all cases a special train, and to record its position at each instant. The train should then be placed in corresponding positions, at rest, to obtain the diagram of static strain. This will be difficult to accomplish, but the results of any other method are open to serious question. Mr. Seamán.

It would also seem preferable to obtain a large number of tests upon a few selected bridges of varying spans, rather than upon a great variety of bridges, since the varying peculiarities of each bridge may affect results, which would be erroneously attributed to a change of span. Different sets of experiments, with varying designs, might be desired; but they should be kept separate so that the effect of span, or of rate of application, may be clearly traced.

It is also important that the effect of vibration or oscillation should be distinguished from that of impact or sudden application. Unbalanced drivers appear to cause the principal irregularity, and the effect is plainly noticeable when riding upon a badly balanced engine, passing rapidly over a long span.

The effort to deduce an impact formula from the centrifugal force due to the curve of deflection or from the effect of camber, is objectionable, because these conditions vary with different bridges, and, in some instances, are framed out altogether. The speaker entered into the theoretical consideration of the subject some time ago and adopted the general formula:

$$I = \frac{C}{\sqrt{L}} + C^1$$

Where I = percentage of increase of strain.

L = distance through which the moving load must pass to produce the given strain.

C and C^1 = constants.

This formula was based upon the assumption that I would vary with the time of application of the load. C^1 is the effect of oscillation, which would be assumed as approximately constant.

In outlining the paper on bridge specifications,* which the speaker

* *Transactions*, Am. Soc. C. E., vol. xli, p. 140.

Mr. Seaman. recently presented to the Society, the value of C^1 was included in the allowable live strain and believed to amount to 30% or more. The remaining values were given in the form of a table, but were slightly varied to conform to a quadrant of the curve of an ellipse. This is found to give a heavy increase for short spans, and the experiments appear to confirm this view. The maximum increase would be upon that span where the deflection would be half the total drop of the load while passing to mid-span. The span would probably be about 5 ft. in length, and the increase might approach 100 per cent. On these short spans the effect of variation in speed would be most apparent. This effect is also noticed when the engine first comes upon the bridge.

The results obtained by the author, which include those due to both impact and oscillation, are interesting and valuable, and it is hoped that they will be amplified with special reference to short spans and sudden applications of the load.

Mr. Berger. BERNT BERGER, Assoc. M. Am. Soc. C. E.—The speaker is inclined to agree with Mr. Seaman in regard to the time of application of the load, but he is not willing to go quite as far in the application of the idea. The author's experiments show that the effect of speed in causing vibration increases with the shorter spans, and Mr. Seaman insists further that if experiments were made on spans 3 to 6 ft. in length, it would be found that the additional effect due to the shortness of the span and the sudden application of the load would amount to 100%, especially with high speeds. The speaker is inclined to think that on 3 to 6-ft. spans the rails alone would support a train going at certain speeds without the assistance of the girders. There is such a thing as the necessity of time for the transmission of stress. It takes a certain time for the members of a structure to feel the application of a load. The speaker does not mean to say that he would bridge 3 to 6-ft. openings with rails alone, because a train might come to a standstill upon them.

Regarding the question of deflections, it is well known that some are apt to give it too much weight; but it is interesting to note that the deflections observed in the author's experiments agree well with the strains as calculated for the loads used. Of course, there was neither time nor opportunity for the experiments to be made on every member in the bridge, but as far as it was possible to cover the subject, the chord strains seem to agree with the deflections very well. As far as the web members are concerned, there are, as might be expected, higher percentages of increase for the strains than for the deflections; because for a maximum deflection there is a different position of the load than for a maximum strain on the web member. The experiments also bring out some interesting points in regard to the distribution of the strains on different parts of the same member. In the chords, for instance, where they consist of eye-bars, the strain seems to be quite evenly distributed, as it also is in the

hip suspenders, where they are eye-bars, and the floor is suspended. But where they are riveted and the floor beams are riveted to them in turn, it is found that in one case the outer portion carries 40% and in another case 70% of that carried by the inner portion. It would be interesting to know at what height above the floor the apparatus was applied, and whether the distribution of the strain would vary with the distance from the point of application of the load. Mr. Berger.

With all the satisfaction which can be derived from the author's experiments as to the behavior of bridges under strain, it is somewhat depressing to note that structures which are considered rigid —rigidity being the aim and object of modern bridge design—show no more freedom from vibrations than structures which the author characterizes as flexible.

Of course, it is practically impossible for one man to cover exhaustively so wide a field of investigation in one series of experiments; but the author has applied himself to his work with great care, and has given by far the most important contribution to the study of the action of bridges under moving loads. The speaker does not believe, however, that by the methods used by the author the true effect of speed alone can be said to be determined independently of other influences. He agrees with the author, however, in the conclusion that the effect of speed alone is of no practical importance, except, perhaps, for floor beams and stringers. The accidental, and more or less unavoidable, inequalities in the track, and the imperfections in the rolling stock, play a far more important part. It is very difficult to separate their effect from that of speed alone, and this effect will, of course, be subject to considerable variations with varying speeds.

T. KENNARD THOMSON, M. Am. Soc. C. E.—Although bridge building has been reduced to an exact science, yet for years engineers, in proportioning the parts of bridges, have been guided by two considerations: *First*, by some German experiments, which were very good in their way, but which by no means covered the ground; and, *second*, by what is politely called judgment, but which is really nothing but good guessing. With this in view, when discussing Mr. Seaman's paper,* the speaker suggested that any engineer who would make an elaborate set of experiments on bridges, as they actually are, would receive the thanks of the profession. The author has anticipated this hope by making a very good start. The speaker thinks that much more satisfactory results could be obtained by many experiments carried on under the direction of one capable man, than by the spasmodic efforts of a number of men, each of whom would have more or less divergent methods, and that the best way to accomplish this is to secure by subscription a fund to enable such a man to devote several years to this work. As the entire profession is to be benefited, each member of it should be willing to contribute. Mr. T. K. Thomson.

* "The Launhardt Formula, and Railroad Bridge Specifications." *Transactions*, Am. Soc. C. E., vol. xli, p. 198

Mr. G. H. Thomson.

G. H. THOMSON, M. Am. Soc. C. E.—This paper has introduced a subject interesting to many engineers, and reminds the speaker of his own gauge studies, about fourteen years ago, of train action upon old railroad bridges of twenty-five years' service, combining the results of gauging with stress computations and examinations of the material of the old bridges after removal. From about this time he began to design railroad bridges with reference to the speed of the loading.

The sudden imposition of a load on a truss-member, with concomitant and responsively rapid lengthening, and a sudden removal of load, with like rapid shortening, may induce in some bridges more truss motion than is desirable.

The undulations noted by the author can be found in familiar examples. A rubber band, with a suspended ball, for instance; or an experiment illustrating the effect of time loading can be made with a butcher's spring scales. Place a 10-lb weight on the hopper, and the gauge registers, say, 13 lbs., pulsating back to 8 lbs., and so on, up and down, until quiescent. Place now 10 lbs. of fine shot in a funnel, controlling with the finger, the discharge of the shot from the spout so that it can be slowly delivered to the hopper; the gauge will then record slowly 1 lb., 2 lbs., and so on, up to 10 lbs., but no more.

Some bridges pulsate (much after the manner of scales), during the passage of trains and for some time afterward. Bridges are not designed for purposes of weighing, however, and bridge-members are not coiled springs.

The results shown by the author, due to the lengthening and shortening of members, are about what is to be expected from the nature of the loading, taken together with the type of trusses, length of panel, etc.; and the observed undulations will vary much as the sectioning of the members vary under the variety of specifications now in vogue. For illustrations of this, take the diagram Fig. 15, and select any bar, say a (to load which consumes nine seconds of time), and consider the upper eye fixed, and the lower eye movable; and further, referring to the diagram, Fig. 16, it will be seen that the lower eye is shown in its movements as follows: Line 1 is the place of the eye corresponding to a bridge upon its false works; the false works being removed, the eye will descend to line 3, which position is constant for the structure, except at such times as train service may call the bar into action, when the eye, under normal conditions of train-loading, coupled with adequate and judicious bar section, will descend to line 6; during the passage of the train, the eye will move between the limits of lines 3 and 6, with much of this movement between lines 4 and 5; and as the train leaves, and after it is entirely off the bridge, the eye movement gradually lessens, until, the truss motion ending, finds the eye at its constant position at line 3.

In case the imposition of the load is irregular in spacing and in-

TABLE No. 6.

Location of diagonals.	*Case No. I.* One single moving load.					All Panels Loaded by Live Load up to Diagonal Considered and Spaced 25 Ft. = 1 Panel.												
						Case No. II.					*Case No. III.*			*Case No. IV.*			*Case No. V.*	
	Stretch of each diagonal.	Time of loading.	Stretch per second.	Time for release of loading.	Release* of loading per second.	Stretch of each diagonal.	Time for loading.	Stretch per second.	Time for release of load.	Release* of load per second.	Live load / Total area $\times \frac{1}{E} = X$.	Total stretch in inches, 360 ins. $\times X$.	Stretch in inches per second of loading.	Live load / Total area $\times \frac{1}{E} = X$.	Total stretch in inches, = 360 ins. $\times X$.	Stretch in inches per second of loading.	Live load / Total area $\times \frac{1}{E} = X$.	Total stretch in inches = 360 ins. $\times X$.	Stretch in inches per second of loading.
		Secs.	Ins.	Secs.	Ins.		Secs.	Ins.	Secs.	Ins.		Ins.	Ins.		Ins.	Ins.		Ins.	Ins.
a.....	For all diagonals— Stretch one way = 0.12 in. " both ways = 0.24 in.	9	0.0133	1	0.1200	For all diagonals— Stretch one way = 0.12 in. " both ways = 0.24 in.	9	0.0133	9	0.0133	$\frac{1}{4430}$ L	0.0791	0.0090	$\frac{1}{4030}$ L	0.0892	0.0099	$\frac{1}{3660}$ L	0.0984	0.0109
b.....		8	0.0150	2	0.0600		8	0.0150	9	0.0133	$\frac{1}{4470}$ L	0.0802	0.0100	$\frac{1}{3970}$ L	0.0905	0.0113	$\frac{1}{3710}$ L	0.0970	0.0121
c.....		7	0.0172	3	0.0400		7	0.0172	9	0.0133	$\frac{1}{4460}$ L	0.0818	0.0117	$\frac{1}{3930}$ L	0.0916	0.0131	$\frac{1}{3790}$ L	0.0950	0.0135
d.....		6	0.0200	4	0.0300		6	0.0200	9	0.0133	$\frac{1}{3940}$ L	0.0910	0.0152	$\frac{1}{3640}$ L	0.0990	0.0165	$\frac{1}{3660}$ L	0.0985	0.0164
e.....		5	0.0240	5	0.0240		5	0.0240	9	0.0133	$\frac{1}{3180}$ L	0.1130	0.0226	$\frac{1}{3106}$ L	0.1160	0.0232	$\frac{1}{3286}$ L	0.1095	0.0218

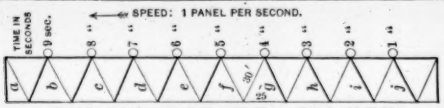


FIG. 15.

NOTE.—E = Modulus of elasticity = 30 000 000. *Shortening due to release.

Cases Nos. I and II.—Bridge has no weight. Diagonals stretched by the load (at 10 000 lbs. per square inch), so that the extension of each diagonal ($\frac{1}{3000}$ of length), is an equal amount.

Case No. III.—Dead and live load considered. Sectional areas of diagonals proportioned at 10 000 lbs. per square inch for live and dead loads.

Case No. IV.—As Case No. III, but sectional areas proportioned at 15 000 lbs. per square inch for dead load.

Case No. V.—As Case No. IV, but sectional areas proportioned to G. H. Thomson's specification of 1894 for the live load.

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tensity, coupled with flat wheels, hammer blows, etc., and, in case the elastic relations inhering in the trusses are lacking in co-ordination (a fault of the design due to unsuitable length of panel, blind adherence to current specifications as to unit stresses, inattention to change of length of members, etc.), the eye will move beyond line 6 to line 7; and upon removal of the live load, sometimes, will return to line 2. This distance, lines 6 to 7, and also lines 2 to 3, can be stated as the "abnormal" travel of the eye; the distance, lines 3 to 6, the "normal"; in which "normal" may be understood to represent a measure of limitation determined by the engineer from his knowledge of the forces he handles, in reference to, and resident in, the material he uses; say its qualified powers of elastic resistance.

A bridge may be heavy, or over stocked with section, and therefore of low unit-stresses, and yet be "abnormal" in action; again, light

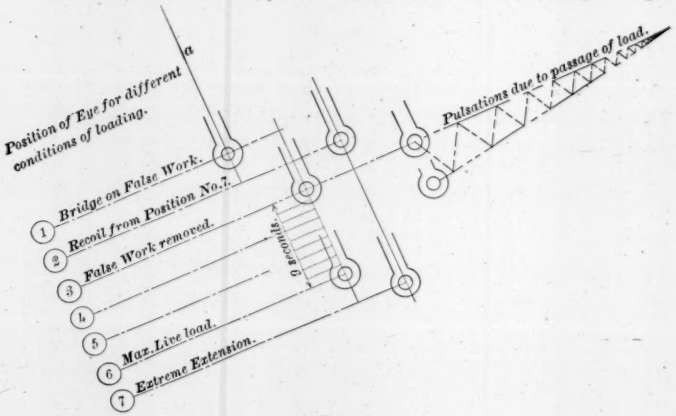

FIG. 16.

bridges are standing to-day with excessive stresses, and are entirely "normal" in action under heavy traffic, covering all kinds of rolling stock at various speeds.

The question may here be asked: For wear and tear of railroad traffic, is a bridge which is dimensioned, say, for 8 000 lbs. per square inch only, and "abnormal" in elastic action, more to be desired by railroad interests than one of, say, 12 000 lbs. per square inch, or more, with "normal" elastic action? The speaker has some very decided views on this point.

The speed of trains enters largely into matters "abnormal." Viewed briefly, there is a quantitative triad, *viz.*, train weight, train space and train time, as cause; and producing, as effect, extension, which is a manifestation of the totality of molecular action. Between cause and effect there is a relation of transmission.

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In a bar there is, in a given unit of time, the long spaces of train weight and the short bar spaces, the result of molecular motion. The ratio between the train space and bar-extension space involves the matter of "normal" or "abnormal" truss action.

Whatever may be the order of arrangement of the grouping of the elemental units of mass in a bar; whatever may be their mode of assembly; whatever may be the theory of their mutual coherence; whatever may be the quality of their being and action, we are logically justified in assuming that all these relations obtain under a rule of existence, *i. e.*, a physical law of coherence of association of units, constant in its operation, within the limits of the scope of the law. Any disturbance of this coherence of the elemental units of mass involving the change of interaction between the units, amounting to an infringement of this law, finds a penalty in compromised molecular integrity. The physical injury to the bar *a*, due to extension from line 1 to line 2 (occurring but once and being very small), can be disregarded. The extension from line 3 to line 6 can be kept within the limits of the provisions of the law.

One of the products of the changed inter-action of the elementary units of mass (say, of a tension bar), finds its expression in the phenomenon of elongation, the magnitude of which can be visualized and measured, if of appreciable amount. If it is desired to call into service the force stored up in a mass, and therein resident by virtue of the law of coherent association, keeping within the outside limit of the law, the units must be given plenty of time to adjust themselves to the changed inter-action; or rather, it is not so much a matter of time, or of how fast the loading, or how slow the stretching of the bar, as of the relation of time between the loading and stretching.

(1) Train action is to be met by molecular reaction.
(2) Train spacing " " " spacing.
(3) Train motion " " " motion.
(4) Train momentum " " structural momentum.
(5) External relations " internal relations.

That the ratio of space covered by metallic linear motion to space covered by train motion varies much in different parts of a truss, is shown on the diagram, Fig. 15, where uniform loads are used. The diagram is merely for an illustration.

Case I.—All diagonals under the live load have an extension of 0.12 in. The extensions per second range from 0.0133 in. for bar *a* to 0.024 in. for bar *e*, a difference of 0.0107 in. The shortening of the bars *a* and *e* per second ranges from 0.12 in. to 0.024 in. Bar *a* receives its severest test in unloading.

Case II.—In this case the extensions are the same as in Case I, save in the shortening due to releases of load which = 0.0133 in. per second for each bar.

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Case III.—In this case dead as well as live load is considered at 10 000 lbs. per square inch, and is fairly representative of many bridges in service. The extensions range from 0.0090 in. for bar *a*, to 0.0226 in. for bar *e*, per second. These extensions are computed, but when gauged, another set of extensions, depending upon the speed of the trains, will obtain.

Case IV.—With the dead load at 15 000 lbs. per square inch, and live load at 10 000 lbs. per square inch, the extensions gauge nearer to computation than in Case III.

Case V.—The extensions gauge and compute the same. Flat wheels, hammer blows, unequal loading and spacing have no measurable effect. There are many bridges of from 125 to 240-ft. span, built in accordance with these specifications. Some 200-ft. spans are of narrow depth; all are free from undue structural motion.

Taking up now Case III, the stretch of the whole bar, *a*, in one second of time = 0.0090 in., the bar being 30 ft. long. The stretch of 1 ft. of bar for one second of time $= \frac{0.0090 \text{ in.}}{30} = 0.00030$ in. While the bar is extending this amount for 1 ft. of its length, the train has passed 25 ft. or $\frac{0.00030 \text{ in.}}{25} = 0.000012$ in. = the extension of 1 ft. of bar while the train moves 1 ft.; and $\left(\frac{0.000012}{12.000000}\right) = \frac{1}{1\,000\,000}$, that is, the train moves 1 000 000 times as fast as the movement of metal in the bar.

First.—Comparison of Case III with Case V. $\left(\frac{\text{Metal motion}}{\text{Train motion}}\right) = R$.

Speed = 25 ft. per second.

	Range.	
Case III.—Bar *a*	1 000 000	
" *V.*— " *a*	882 352	
" *III*— " *e*	387 161	
" *V.*— " *e*	416 236	
Case III.—Bar *a*	1 000 000	
" " — " *e*	387 161	
	613 839	= Difference of range between *a* and *e*.
Case V.—Bar *a*	882 352	
" " — " *e*	416 236	
	466 116	= Difference of range between *a* and *e*.
Case III.—Bar *a* =	1 000 000	
" *V.*— " *a* =	882 352	
	117 648	= Difference in range.
Case III.—Bar *e* =	387 161	
" *V.*— " *e* =	416 236	
	29 075	= Difference of range.

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Case V absorbs train motion faster into bar *a*.
" " " slower into bar *b*.

Suppose, now, that for Cases III and V the speed be increased threefold, or to 75 ft. per second. For Case III the same range will obtain for all speeds, by computation, but it does not follow that the gauged extensions for the high speed will be the same as for the low speed.

For Case V the live loads (1894 specifications) are found by the formula for permissible unit stress $= F\,(T - l)\,a$
in which F = general factor = 7 500 lbs,
T = time for imposition of load in seconds,
a = ratio factor = 500.

Hence for bar *a*, at 9 seconds, $P = 9 \times 500 = 4\,500 + 7\,500 = 12\,000$ lbs. per square inch.

And for bar *e*, at 5 seconds, $P = 5 \times 500 = 2\,500 + 7\,500 = 10\,000$ lbs. per square inch.

Second.—Comparison of Case III with Case V at high speed = 75 ft. per second.

	Range.
Case III.—Bar *a* =	1 000 000
" *V.*— " *a* =	977 900
" *III.*— " *e* =	387 161
" *V.*— " *e* =	556 586

Case III.—Bar *a* =	1 000 000
" " " *e* =	387 161
	613 839 = Difference between *a* and *e*.

Case V.—Bar *a* =	977 900
" " " *e* =	556 586
	421 414 = Difference between *a* and *e*.

Case III.—Bar *a* =	1 000 000
" *V.*— " *a* =	977 900
	22 100 = Difference between *a* and *e*.

Case III.—Bar *e* =	387 161
" *V.*— " *e* =	556 586
	169 425 = Difference between *a* and *e*.

Third.—Comparison between Cases III and V:

Difference of range between *a* and *e*

at 25 ft. per second	613 839	466 116 = 147 723
" 75 " "	613 839	421 414 = 192 425
	000 000	24 702

Mr. G. H. Thomson.

By adjusting the speed or rate of metal motion, making it slower in certain members and faster in others, co-ordinated elastic relations in concordance with the assumed stresses result.

A number of covered, through Howe bridges, of thirty years' use, came under the speaker's notice, wherein the rods, three at each panel point, and of the same size throughout the bridge (with the actual dead load and a live load of 2 000 lbs. per lineal foot), stresses were

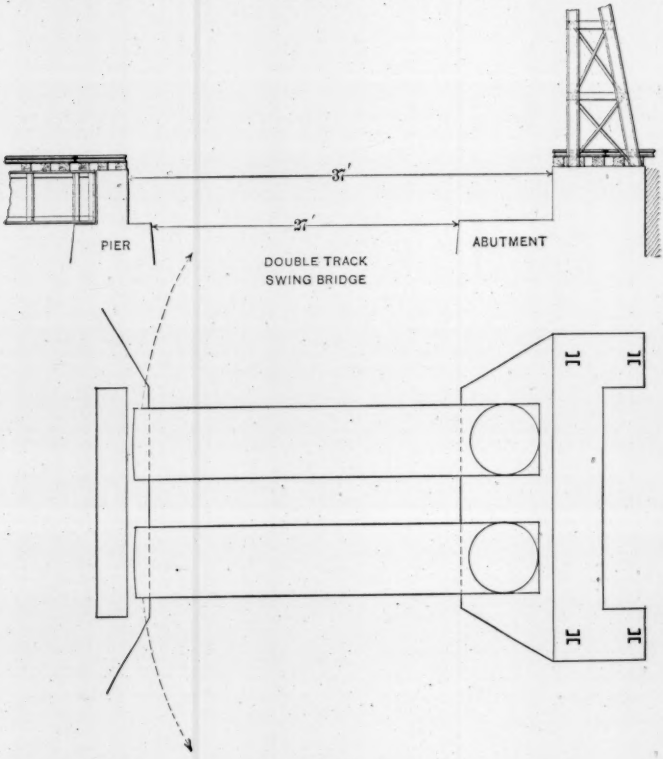
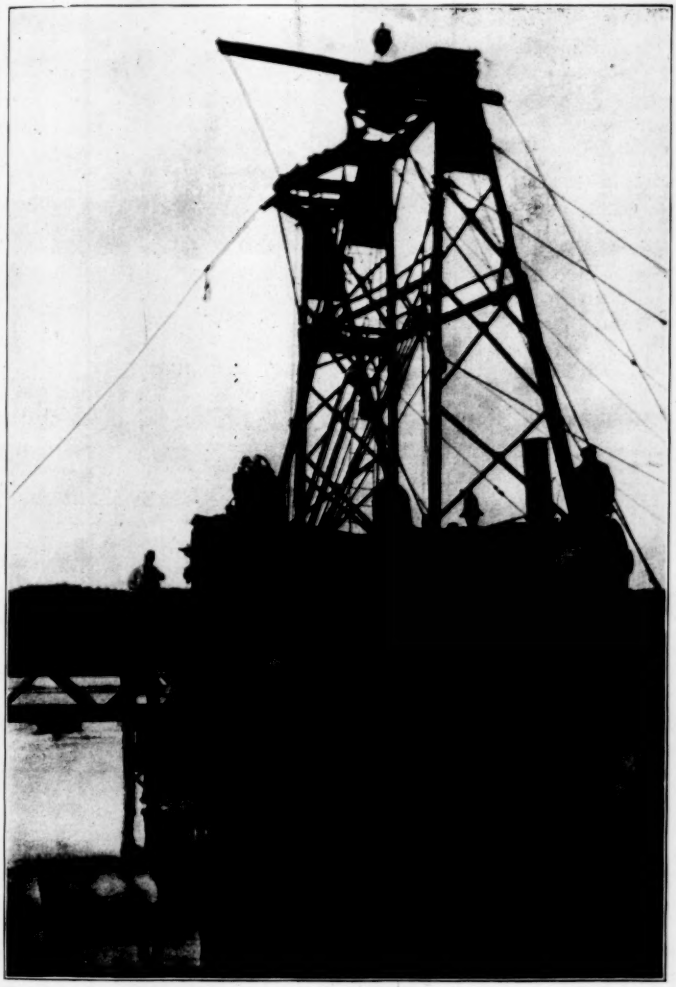


FIG. 17.

found of 30 000 lbs. per square inch for the end rods, and 15 000 lbs. per square inch for the center rods. These old bridges furnish a record of persistence of structure due to the relation of metal motion to train motion. The non-failure of the end rods at stresses beyond the elastic limit can be explained by the fact that the motion induced in the metal was a matter of the slow delivery of the

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLI, No. 849.
THOMSON ON TRAIN-LOAD EXPERIMENTS.



Mr. G. H. Thomson.

train-load to the rod. The end rods of such old bridges often show better fractures than the center rods. Tests of old rods often give high ultimates and elastic limits, about the same for center as for end rods, but the center rods break more like "pipe stems" than the end rods of the same bridge; or, a rod will withstand a live load for a maximum number of applications, if the load is slowly imposed. If a rod has already a constant or dead-load stress, still, the live load, if imposed slowly, will be better resisted by the rod, *i. e.*, if the speed of the metal motion is favorable when compared with the speed of the loading.

The great length of the rods of "through" Howe bridges, doubtless, is favorable for the reception of the loading.

The author appears to attach considerable weight to the matter of deflection. Viewed as a factor involving structural integrity of truss bridges, the magnitude of the central deflection may be of value; but with plate-girder bridges the speaker does not think the fact of deflection is of much value. With short-span deck-plate girders of light section, the amount of the deflection at high speed is frequently one-half that obtained by low speeds or with engines at rest. A rapid passing of the engine over a light plate-girder reminds one of skating over thin ice. The speaker recalls a case where a canal boat, knocking a track stringer out of a bridge, the bridge was used for several days without any stringer whatever. This was a panel of 10 ft. In another case a deck-girder of 10-ft. span had been removed (leaving the rails and ties in place), preparatory to substituting a new bridge. A belated fast train, paying no attention to signals, ran over the opening, on the rails, *i. e.*, with no bridge to sustain the train. In yet another case, which is illustrated by the diagram, Fig. 17, a double-track opening of 37 ft. was covered by two deck-lattice girders, one for each track. The girders were turned upon a center at the abutment end, while the pier end was sustained by guys held from a tower resting upon the abutment. The girders, for one track, were in position for traffic, while for the other track the girders were "swaying" 90° from the track, leaving a clear opening of 37 ft. An express train approaching applied the brakes at a point 500 ft. from the draw. The engine jumped the opening as shown by the photograph, Plate XI, the forward truck clearing the opening entirely. Also, in trusses, some counters have been observed in which the last wheels of a passenger train produced twice the extension made by the engine drivers.

Regarding vertical truss motion ("jumping" in particular, which is most easily felt on through-bridges while standing thereon during the passage of a fast and light train—this is not to be confounded with the vertical motion of mere deflection), the web members, principally, govern its extent and character. Some time is consumed to absorb the train weight into the chords, and hence they are not given to the over-

Mr. G.H. Thomson.

extension lines 6 to 7 of the diagram, Fig. 16. If a train passes two panels in, say, one second, the extension of the diagonal is measured by y. Resolved into z it is not easily appreciated; into x it is visible. For the two panels of a diagonal system the vertical motion $= 2\,x$, of which there is one x for each diagonal (see diagram, Fig. 18); and $2\,x$ for diagonals, and one x for the vertical post (see Pratt system, of diagram, Fig. 19).

The term "structural motion" refers to the motions induced in structures by the moving loads.

Structures without some motion are inconceivable, but undue structural motion may be controlled and reduced to reasonable limits without increasing the cost.

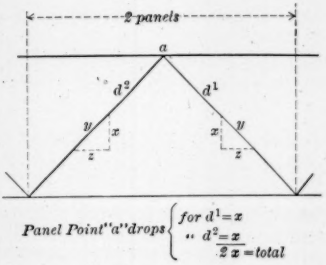


FIG. 18.

A bridge, in strength, must meet all statical conditions; but is this enough? The fractures, etc., of the metal of old bridges, with motion, do not compare favorably with those old bridges that were free of motion.

To make bridges free of motion, a number of points require recognition: The type of truss, the length of panel (with its value to receive equally and distribute or disperse the loading), the elastic resistances of material, the positive momentum of trains with negative momentum of bars, etc.,—all of these co-ordinated to time ratios, *i. e.*, speed of trains—or, train motion, *versus* molecular motion.

A structure free of motion, void of over-strain, is suggestive of permanence and persistence, and a railroad bridge that is strong, as viewed from a statical point, if free of structural motion, invites confidence and holds it. Many bridges having spans of from 125 to 240 ft. of Class V specification, are in service, where the deflection is only about $\frac{1}{7000}$ of the span, though this ratio in itself is of no particular value.

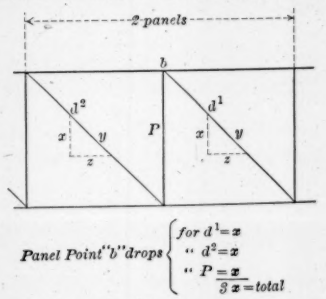


FIG. 19.

It is not necessary to put so much metal in members that consume much time to load, *i. e.*, the bottom chords. A through-bridge designed August 15th, 1894, under Class V specifications (not built), for Augustus Mordecai, M. Am. Soc. C. E., for Erie engines, weighed, as per detail plans, 123 tons for a clear span of 150 ft.

CORRESPONDENCE.

J. B. JOHNSON, M. Am. Soc. C. E.—The author deserves, and will doubtless receive, the thanks of the profession for his remarkably thorough experiments and his exhaustive study of the same. While his results had all been anticipated and provided for in American bridge practice, yet there was no positive proof that the assumed limits of impact effect were very nearly correct. Mr. Johnson.

Many engineers have had a lurking fear that there might be immediate or cumulative stresses far beyond those ordinarily provided for, and these experiments will reassure them that the present usages in the United States, as to working loads, are fully justified.

The discovery of certain large secondary stresses in plate girders, also, when eccentrically loaded, as on shelf angles, is only what an analysis of such cases would have shown. This fact simply emphasizes the necessity for the complete analysis of engineering structures, in all their parts and details, for the actual conditions of loading, which is now receiving attention in the leading engineering schools, as well as in the computing offices of practitioners. It is evident that the limit of fruitful mathematical analysis in engineering design has not yet been reached.

F. E. TURNEAURE, Assoc. Am. Soc. C. E.—Mr. Seaman's criticism in regard to comparing the observed results with the computed, would appear to be hardly pertinent. Computed results are used only in the discussion relating to the effect of speed alone, and then in but three out of the eight bridges so discussed are the computed results of any significance. In the other five cases a special train was used, and was moved over the bridge at so low a speed as to give a strain diagram unaffected by dynamic action, and which could therefore serve as a standard. For convenience, however, all stresses were referred to the computed values, and the significance of the diagrams in Figs. 12, 13 and 14 is not in the absolute values of any of the ratios, but in the horizontality of the lines, showing that the deflections and stresses were practically the same at all speeds. Whether the stresses of the various diagrams from this special train coincided or not with the computed values, is of no consequence, as the observed values are themselves directly comparable. Mr. Turneaure.

The purpose of the theoretical discussion of the effect of speed of application was, not to derive a working value for it, but to show that for ordinary spans it could be of no practical consequence. Camber may of course remove it entirely, or, in fact, give it a negative value, all of which is quite immaterial in practice. The author aimed to show that, both theoretically and experimentally, practically all the increase in deflection, or stress, in the spans tested could be attributed to vibration and oscillation. If in Mr. Seaman's formula it is intended to include the effect of speed of application (that which would still remain were the load perfectly balanced) in the first term, and

Mr. Turneaure. all other effects in the second term, in the author's opinion the first term should be made equal to zero, at least for all spans above 25 ft. in length, and the second term made to vary inversely with span length. In very short spans, such as 5 ft., the effect of speed would theoretically be great if the beams were shallow, but would be a maximum, not at the center, but some distance beyond. In a 5-ft. span for example, with a depth of beam equal to one-tenth the span, there would be, according to Zimmerman's formula, an increase of 30% in the central deflection, and more than 100% for a point two-thirds across the beam. For still shorter spans the central deflection becomes less than for static load, but near the abutment it would be many times increased. All of this assumes absolutely rigid abutments, track and rolling load, but the deflections involved in such very short spans are so small that a very slight yielding would entirely vitiate any calculations such as the above. Cases of trains passing safely over very short spans with no other support than the rails, as mentioned by Mr. G. H. Thomson, are instances to the point, but the author ventures to say that in such cases the abutment, or other "landing place," received a pretty severe pounding. If the load is constrained to move in a straight line there could be no effect of speed either in adding to the load or in reducing it. In the actual case, both girder and supports are elastic. The effect of deflection of the girder is to increase the pressure, while the effect of elastic supports is to diminish it. In the author's opinion the former is too small to take into account on any but the very shortest spans, and in those there would doubtless be sufficient elasticity in the supports to more than counterbalance this effect. The practice of supporting short girders on wooden wall plates would seem to be a good one, as tending to relieve both girder and masonry.

A matter which should unquestionably be taken account of, and which is clearly brought out by both deflection and stress diagrams, is the increase of stress in short girders caused by the locomotive, and which increases rapidly with decrease of span.

Answering Mr. Berger's question in regard to the stresses in riveted hip verticals, the author would say that the stresses were determined at points close to the floor beams. Time was not available for a more extended series of experiments along this line.

In regard to the effect of the vibration, or "jumping," on truss members, the experiments do not bear out Mr. G. H. Thomson's statement that such effect is mostly in the web members, and that the chords receive their stress too slowly to be much affected thereby. While the horizontal component of the web distortion may not be readily appreciated by an observer standing on the vibrating truss, the variation in chord stress with truss vibration is made very evident by watching the action of the extensometers at such times. The close agreement in the relative variation in chord stress, and vibration of the entire truss, in a large number of tests, would appear to be a good indication of the real state of things.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 850.

THE DETERMINATION OF THE SAFE WORKING STRESS FOR RAILWAY BRIDGES OF WROUGHT IRON AND STEEL.

By E. HERBERT STONE, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

WITH DISCUSSION.

INTRODUCTORY.

From the date of the first introduction of railways, it was for many years the custom, in the design of a railway bridge, to treat the Moving Load of the train in precisely the same manner as the Fixed Load of the structure itself. The load due to the weight of the train was simply added to the load due to the weight of the structure (with flooring, ballast, permanent-way, etc.), and the section of iron to be used calculated for a working stress of about one-fourth of the assumed breaking stress. This, for wrought iron in tension, allowed a nominal unit stress of 5 tons per square inch.

Later on, text-books, dealing theoretically with the subject, taught that the effect produced by Live Load was just double that produced by Dead Load, and recommended that the two kinds of load should be treated separately; a factor of safety of 6 being used with the former and 3 with the latter. Taking the breaking stress of wrought iron

in tension as 21 tons per square inch,* the safe working stress per square inch was given as—

Dead Load.....7 tons.
Live Load.....3½ "

It was not recognized that the theoretical "Live Load," on which these considerations were based, was one which was never dealt with in practice, and was, moreover, essentially different in its action and mode of application from that due to a locomotive at speed. Still this system showed a great advance on what had gone before, and had the effect of directing attention to the matter. The result was that, although engineers generally regarded this as a theoretical rule to which it would not be prudent to work fully in practice, it became common to limit the stress on tension bars to 4 tons per square inch for live load, and it was generally acknowledged that for dead load the stress per square inch in tension might safely be taken at 6 tons. The Board of Trade limit of 5 tons, however, precluded any improvement in the latter direction.

This may be regarded as the first recognition of the principle that Moving Load may be equated with Fixed Load by the use of a suitable coefficient; but at this stage the coefficient was supposed to be always the same, irrespective of the relative amounts of the two kinds of load to be dealt with.

As wrought-iron bridges of large span became more common, engineers grew to be familiar with the fact that, with the comparatively great mass of metal in a large-span bridge, the effect of a train is relatively less than with a small-span bridge. In view of this fact it became usual in the practice of some of the leading bridge engineers to adopt a scale by which the unit stress allowed was made to vary with the span. It was thus acknowledged that the effect of Moving Load as compared with Fixed Load was not always the same, but became relatively less as the proportion of Fixed Load to Moving Load became greater.

The principle of graduating or apportioning the unit stress to the relative amount of Moving Load work to be done soon became gener-

* For the purposes of this paper it is assumed that the static breaking stress per square inch, for material as ordinarily used for bridge girders, may be taken as 21 tons for wrought iron and 27 tons for steel. Also, that the safe working stress may be taken as one-third of the breaking stress. Hence, for an entirely static or fixed load, the figures adopted are as follows:

	Iron.	Steel.
Breaking stress in tons per square inch.....	21.00	27.00
Safe working stress in tons per square inch.....	7.00	9.00

ally recognized; and in America the application of this principle to practical purposes received the most careful study by many of the bridge engineers of that country. Under the special allotment system, as developed by American engineers, the members of a bridge truss are grouped or classified according to the relative amount of Moving Load work they have to bear, depending on the span of the bridge and the position of the member in the truss. To each group is then assigned a percentage by which the nominal stress (treated as static) is to be increased to obtain the equivalent effective working stress; the permissible stress per unit of area remaining constant. In more recent specifications similar results are obtained by using the nominal stress (treated as static), and allotting to each group a different permissible stress per unit of area.

Meantime the results of the magnificent series of experiments by Wöhler and Bauschinger had been ably dealt with by Launhardt, Weyrauch, Gerber and others, and bridge engineers began to consider the cumulative effect of frequent repetitions of the Moving Load.

Thus, while forty years ago a ton of Moving Load received no more consideration than a ton of Fixed Load, its disposal now is recognized as involving very complex considerations. It is found that for any member of a bridge the immediate effect of the application of a ton of Moving Load is always greater than that produced by a ton of Fixed Load, and that the degree or percentage by which it is greater varies for different ratios of Moving Load to Fixed Load. Similarly, that for repeated loading and unloading, the ultimate cumulative effect, on any member of a bridge, of a ton of Moving Load, is always greater than the immediate effect, and that the degree or percentage by which it is greater varies for different ratios of the initial stress or Fixed Load.

The Nature of the Effects.

Definitions.—The terms "Dead Load" and "Live Load" have already been appropriated in a certain definite sense by the text-books; and in this sense have a special relative value, the effect of Live Load being exactly double that due to Dead Load. For the sake of distinction therefore in this paper the terms "Fixed Load" and "Moving Load" will be used with meanings respectively as follows:

Fixed Load.—By this term is to be understood a load which may for practical purposes be considered absolutely constant, subject to no

movement, vibration or variation. The self-supported weight of the structure itself is thus throughout this paper treated as "Fixed Load" and also any other load supported by the structure which is stationary or which generally remains in position and is not frequently applied or removed. Thus, for example, the weight of the girders of a bridge would be treated as Fixed Load, so would also the ballast, sleepers, permanent-way, etc.*

Moving Load.—By this term is to be understood a load which is subject to variation or which is frequently applied and removed. The effect of the violence or vibration with which the application of the load is accompanied is considered as an enhancement of the effect due to the mere weight of the load. Thus a dead engine, pulled over a bridge by a rope from a crab winch beyond, would be treated as Moving Load; the same locomotive traversing the bridge at full speed would be considered as having an enhanced effect due to the violence and vibration with which its application was accompanied.

The "Live Load" of the Text-Books.—In the case of a girder, the "Live Load" of the text-books would mean a load suddenly applied in a vertical direction, but without impact. This might be represented by a load just touching the upper surface of the girder, but having its weight entirely supported by a cord. If that cord were instantaneously severed, the load would then act as "Live Load" in the ordinary text-book meaning of the term. The same load, unsupported by the cord, and having its whole weight resting on the girder without vibration, would represent "Dead Load."

It can easily be shown that the effect on the girder of the "Live Load" of the text-books is theoretically double the effect of the corresponding "Dead Load."

Live Load Contrasted with Train Load.—Those writers who apply the theoretical "Live Load" of the text-books to the solution of this problem, assume that the effect produced by the train traversing the bridge in a horizontal direction may be taken as the dynamic equivalent of a load suddenly applied in a vertical direction; and that the faster the engine moves, the more nearly does it become "Live Load," in the text-book meaning of the term.

* For the purpose of experiment on the relative effect of Fixed Load and Moving Load, an engine standing on a bridge is considered as Fixed Load, the same engine traversing the bridge at full speed being taken as Moving Load.

In certain cases with alternating stresses the weight of the structure or a portion of the same may become "Moving Load," *i. e.*, a load applied and removed each time an engine traverses the bridge.

It will, however, be evident on consideration that these two modes of applying the stress are in every respect essentially different. In the case of the "Live Load" of the text-books a load is applied vertically by the steady action of gravity, while in the case of the Moving Load on a railway bridge there is a locomotive engine running horizontally by means of its own self-developed power, with all the plunging and peculiar vibration contingent on the special construction of the machine, the irregularities of the permanent-way, and the deflection of the girders.

An engine standing at rest on a bridge is certainly "Dead Load" in the ordinary text-book meaning of the term; but when that engine begins to move, the effect on the bridge, due to gravity, is reduced instead of being increased. If the effects of vibration and deflection be neglected, and the engine be supposed to run with absolute smoothness, it is clear that the faster the engine moves in a horizontal direction, the less will be its effect in a vertical direction, and that if the speed became infinite the effect due to gravity would become nothing.

The only way in which a train passing on to a bridge in a horizontal direction could be made approximately to represent a load suddenly applied in a vertical direction would be to cause the train to run at a great speed on to the bridge and stop suddenly in the middle.

It is further to be noted that whereas the "Live Load" of the text-book rule means a load suddenly applied, but without shock or violence, the Moving Load of a locomotive differs essentially therefrom; its effect being less on one hand, in that it is not suddenly applied, while it is greater on the other hand, in that its application is always accompanied by a certain amount of vibration, which is in some cases of great violence.

It is evident, therefore, that a consideration of the "Live Load" of the text-books will not help in any way toward a knowledge of the effect produced on a bridge by the Moving Load of a locomotive. The two kinds of load are essentially different in their nature and in their mode of application; the effect produced by the locomotive may be greater than that of the theoretical "Live Load," or it may be less, and should it in any case be found by experiment to be the same, this can only be regarded as a coincidence.

Two-Fold Effect of Moving Load.—When a locomotive at full speed traverses a bridge, there is an immediate measurable effect produced,

which is greater than that due to the same load at rest. This immediate extra effect is due to the sudden and violent manner in which the load is applied, the result being an increased deflection of the structure as a whole, accompanied by a general jarring and vibration, which is especially noticeable in the smaller and lighter members.

It has, moreover, been established, by the experiments of Wöhler, Bauschinger and others, that the immediate effect of the Moving Load is by no means the whole effect, but that frequent repetitions of stress have a cumulative effect on the structure, of very great importance; and it has been ascertained that a bar subjected to stress, repeatedly applied and removed, will ultimately break with a load very much less than it would have borne for a few applications only.

Thus there are two distinct effects of the Moving Load to take into account, and it appears necessary to draw special attention to this point, as it is one which is often lost sight of, even by good authorities.

On one hand it is sometimes found that the "Cumulative Effect" is ignored, and it is argued that on a large-span bridge the effect of the Moving Load is practically no greater than that of the same weight as Fixed Load, because the observed extra deflection is but trifling. On the other hand it is frequently found that the "Immediate Effect" is ignored, and the deductions from Wöhler's experiments by Launhardt, Weyrauch and others are discussed as if cumulative effect were the only matter for consideration.

To determine a coefficient for the Moving Load by a study of cumulative effect alone would be to assume the effect on a bridge of an engine traveling at sixty miles an hour to be no more severe than that due to the same engine "dead" hauled over slowly with a rope from a winch beyond the abutment. On the other hand to determine a coefficient for the Moving Load by a study of the immediate effect alone, would be to assume the effect on a bridge of an engine traversing the structure an indefinite number of times to be no greater than that produced by the engine traversing the structure once only.

In this paper, therefore, the effect of the Moving Load will be considered under two heads, viz.: (*a*) Immediate Effect—which is observable every time a train passes over the bridge; and (*b*) Cumulative Effect—produced in course of time by repeated loading and unloading.

Immediate Effect of the Moving Load.

The Measure of Immediate Effect.—If a locomotive be run slowly on to a bridge and there brought to a stand, the stresses produced will be simply those due to the weight of the machine as Fixed Load. If the engine be now made to traverse the bridge at a considerable speed, the immediate effect of the extra stress produced is indicated by an increased deflection of the structure as a whole, and by an increased elongation or shortening of members in tension or compression.

These effects may be measured by suitable instruments, and the immediate effect produced by the locomotive in motion can be compared with that produced by the mere weight of the machine at rest. Thus, for example, if the deflection of a girder with the engine at speed is found to be half as much again as that due to the dead weight of the engine at rest, evidently this same extra deflection could have been produced with the engine standing on the bridge by loading it with pig iron until the weight resting on its wheels had been increased by 50 per cent.* A ton of Moving Load in this case being found to produce on the bridge an immediate measurable effect as severe as one and a half tons of Fixed Load, it is clear that for the purpose of determining the stresses the actual weight of the Moving Load would have to be multiplied by a coefficient of 1.5, to obtain its equivalent in terms of Fixed Load.

Observations in India on Deflection.—Since the year 1879 deflection observations have been regularly made in India by the Government Inspector on the bridges of each new section of railway before it is opened for traffic. These observations are, whenever practicable, made with self-recording apparatus, and are conducted on a uniform system under Government rules. The author has made an analysis of all the results thus obtained for ten years, from June, 1882, to June, 1892. This deals with nearly 1 500 separate observations made by a number of different inspecting officers in all parts of India where railways had been constructed during that period, and with every variety of girder in ordinary use, over about 7 500 miles of new railway. Some of these lines are on the 5 ft.-6 in. gauge, others on the meter-gauge. Some are owned by the Government, others by companies.

It will be seen, therefore, that the results thus obtained represent averages, not only of a considerable number of actual observations,

* This, of course, depends on the assumption that the measurable deflection increases directly as the actual stress.

but also for a variety of different spans and different designs. Further, that the experiments were conducted on a uniform system and under practical conditions by a number of entirely independent observers. No doubt, in many cases the original data may have been but roughly determined, and the observers perhaps sometimes did their work in a careless or perfunctory manner; but any errors due to defective observation are as likely to be on one side as the other, and with the large number of observations on which the averages were taken, and the large number of different observers who contributed to these averages, it is probable that the results obtained fairly represent the tendency of the actual effects produced under ordinary conditions.

Collation of the Results.—In collating and arranging these data the average of all the observations was first taken for each span separately, and the results so obtained plotted by points on the diagram. A note was then made at each point to show the number of observations on which the position of that point depended, and each point in succession was joined to that immediately beyond it by a straight line. On each straight line so obtained a mark was made dividing it into two parts in the inverse ratio of the numbers at its end points. These marks thus again averaged the averages previously obtained, in the ratio of their relative importance. It will be seen, therefore, that under this system each point was given votes (as it were) in proportion to the number of observations it represented, and the position in which the curve was finally drawn was determined accordingly. Through the last point, thus obtained experimentally for the greatest proportion of Fixed Load, the line was continued as an even curve to pass through the point of all "Fixed Load" where the immediate effect of the Moving Load becomes nothing. The result thus obtained is illustrated in the diagram, Fig. 1.

Endeavors were made, by different systems of plotting and in other ways, to discover some law or rule by which the form of the curve might be determined, but without success. A parabola was, of course, first suggested, but was found quite irreconcilable by any reasonable concessions. The result now presented, therefore, embodies no theory; it is merely an even curve averaging the actual results as nearly as practicable.

Results Accepted for Practical Purposes.—To what extent the results now obtained for girders as complete structures apply to individual

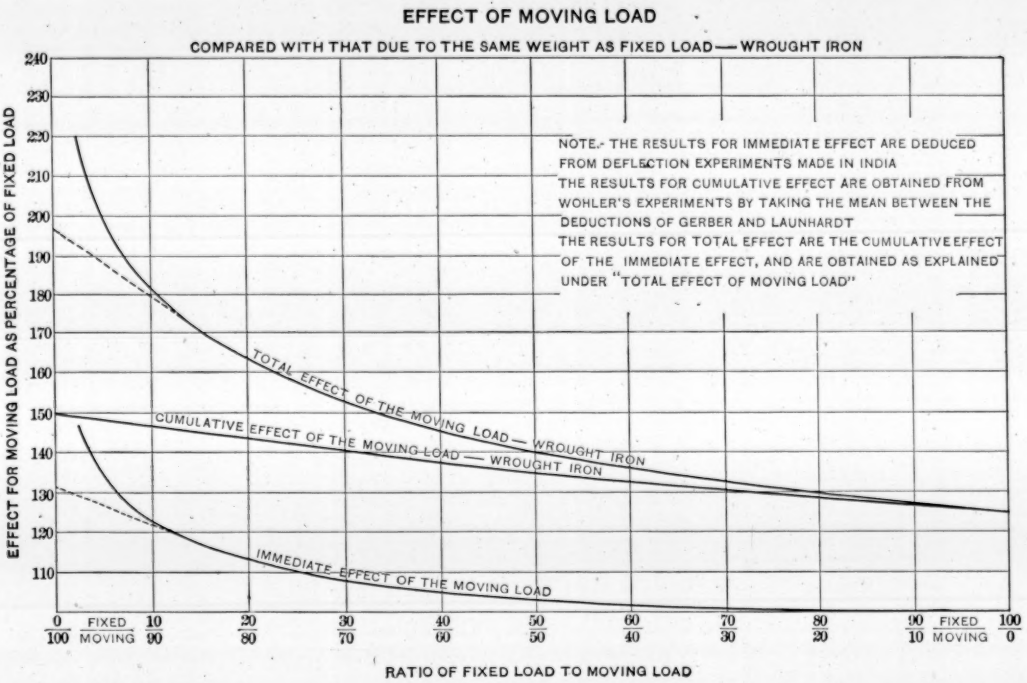
EFFECT OF MOVING LOAD
COMPARED WITH THAT DUE TO THE SAME WEIGHT AS FIXED LOAD — WROUGHT IRON
EFFECT FOR MOVING LOAD AS PERCENTAGE OF FIXED LOAD
240
230
220
210
200
190
180
170
160
150
140
130
120
110
NOTE.- THE RESULTS FOR IMMEDIATE EFFECT ARE DEDUCED FROM DEFLECTION EXPERIMENTS MADE IN INDIA
THE RESULTS FOR CUMULATIVE EFFECT ARE OBTAINED FROM WOHLER'S EXPERIMENTS BY TAKING THE MEAN BETWEEN THE DEDUCTIONS OF GERBER AND LAUNHARDT
THE RESULTS FOR TOTAL EFFECT ARE THE CUMULATIVE EFFECT OF THE IMMEDIATE EFFECT, AND ARE OBTAINED AS EXPLAINED UNDER "TOTAL EFFECT OF MOVING LOAD"
TOTAL EFFECT OF THE MOVING LOAD — WROUGHT IRON
CUMULATIVE EFFECT OF THE MOVING-LOAD — WROUGHT IRON
IMMEDIATE EFFECT OF THE MOVING LOAD
0/100 FIXED MOVING 10/90 20/80 30/70 40/60 50/50 60/40 70/30 80/20 90/10 FIXED MOVING 100/0
RATIO OF FIXED LOAD TO MOVING LOAD

FIG. 1.

truss members can only be determined by an extended series of detailed experiments on individual members in girders of different types; but as the law by which the effects are governed must be the same in both cases, it is probable that the scale by which the value of the coefficient varies, for different ratios of Moving Load to Fixed Load, will be of a similar character; and as the results obtained from the deflection of the truss, as a whole, must to a great extent represent the average for all its members, it will be evident that the error cannot in any case be of great importance.

It is probable that a complete series of experiments, carefully conducted with efficient apparatus, would indicate that each class of members of a triangulated girder has its own proper series of coefficients, and that another series would apply to plate girders. If the lines representing each of these series of coefficients were plotted on a diagram, a maximum and minimum boundary line could then be drawn, enclosing a strip or band within which all the coefficient lines would lie.

Remembering that herein only the coefficient for immediate effect is being discussed—that this has to be combined with the cumulative effect to obtain the total effect—that this total effect is for Moving Load alone, and that to obtain the total effective stress on the bar the Fixed Load is to be added—remembering, further, that the safe working stress on a bar, due to all these effects combined, is only one-third of the ultimate breaking stress for that bar—it will, no doubt, be agreed that the width of the strip or band, containing the various coefficient lines, is not likely to be great enough to affect to any important extent the actual dimensions which would be given to any individual member of a truss as a result of these calculations. Still less would it be likely to affect the approximate formula recommended for general use.

For practical purposes, therefore, and in the absence of more complete data, it is considered that the results of the deflection experiments made in India may be accepted as a sufficiently close approximation to the average immediate effect produced on the members of a bridge truss by a passing train.

The general results for the immediate effect of the Moving Load, as obtained from these experiments, are given in Table No. 1*, for different

*For the purposes of this paper it is assumed that the static breaking stress per square inch, for material, as ordinarily used for bridge girders, may be taken as 21 tons for wrought iron and 27 tons for steel. Also, that the safe working stress may be taken as one-third of the breaking stress. Hence, for an entirely static or fixed load, the figures adopted are as follows:

	Iron.	Steel.
Breaking stress in tons per square inch.....	21.00	27.00
Safe working stress in tons per square inch.....	7.00	9.00

conditions of loading, from "All Moving Load" advancing by 5% differences up to "All Fixed Load." These results are also exhibited in a graphic form in Fig. 1.

TABLE No. 1.—IMMEDIATE EFFECT OF MOVING LOAD. DEDUCED FROM DEFLECTION EXPERIMENTS MADE IN INDIA.

Nature of Compound Load. Ratio Percentage.		Immediate effect of Moving Load compared with that of Fixed Load. Percentage.	Immediate Breaking Stress for the Compound Load. Tons per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	.....	.....	....	.....
2.5	97.5	147.40	14.36	0.36	14.00
5	95	133.78	15.90	0.79	15.10
10	90	122.63	17.45	1.74	15.70
15	85	116.86	18.37	2.76	15.61
20	80	113.06	19.01	3.80	15.21
25	75	110.27	19.50	4.87	14.62
30	70	108.13	19.87	5.96	13.91
33.3	66.7	107.14	20.05	6.68	13.37
35	65	106.63	20.13	7.05	13.09
40	60	105.47	20.33	8.13	12.20
45	55	104.62	20.48	9.22	11.26
50	50	103.87	20.60	10.30	10.30
55	45	103.23	20.70	11.38	9.31
60	40	102.63	20.78	12.47	8.31
65	35	102.10	20.85	13.55	7.30
66.7	33.3	101.96	20.86	13.91	6.95
70	30	101.64	20.90	14.63	6.27
75	25	101.21	20.94	15.70	5.23
80	20	100.82	20.97	16.77	4.19
85	15	100.53	20.98	17.84	3.15
90	10	100.30	20.99	18.89	2.10
95	5	100.13	21.00	19.95	1.05
100	0	100.00	21.00	21.00	0.00

CUMULATIVE EFFECT OF THE MOVING LOAD.

The celebrated experiments of Wöhler, followed by further observations in the same direction by Spangenberg, Bauschinger and B. Baker, have furnished engineers with a mass of data regarding the cumulative effect produced on a structure by repeated loading and unloading.

The experiments, it must be remembered, were conducted with apparatus specially arranged to apply and remove the load in a steady and uniform manner, the effect of shocks and the violent jarring and hammering action, which is observed when a locomotive traverses a bridge at speed, being entirely eliminated. The results of these ex-

periments on cumulative effect are, therefore, exactly what is required to supplement the observations on immediate effect.

The general results are familiar to all engineers who have interested themselves in the subject, and are briefly as follows:

Let t = The greatest stress the bar will bear under a static load. Condition — "All Fixed Load."
(*Tragfestigkeit*, or Statical breaking strength.)

u = The greatest stress of which the bar will bear an indefinite number of repetitions with the load applied and removed. Condition — "All Moving Load."
(*Ursprungsfestigkeit*, or Primitive strength.)

$+ v$ and $- v$ = The greatest stress of which the bar will bear an indefinite number of alternations. The load producing tension and compression alternately; the stresses being equal, but in opposite directions.

a = The actual stress under which the bar breaks—sometimes called the "Working Strength" of the bar.
(*Arbeitsfestigkeit*, or Ultimate working strength.)

Min. S = The least stress to which the bar is subjected.

Max. S = The greatest stress to which the bar is subjected.

$\phi = \frac{\text{Min. } S}{\text{Max. } S}$. This expression also represents the ratio of initial stress or Fixed Load to the Total Load. For example, if a compound load be made up of one-fourth Fixed Load and three-fourths Moving Load, the value of ϕ will be 0.25.

First.—With a Moving Load applied and removed an indefinite number of times, and alternating from zero to a certain fixed quantity, a bar will ultimately break with a load considerably less than that which it would have been able to bear as Fixed Load. In other words, u is always considerably less than t. The ratio of $u : t$ is found to vary with different materials, and some of the experiments show great discrepancies; but, in a general way, it appears that the difference between u and t is greater in steel with a comparatively large percentage of carbon and high static breaking stress, and less with mild steel having a lower static breaking stress. With wrought iron, moreover, it is generally less than with mild steel.

The actual relative value of u, as compared with t, may be taken roughly as varying from $u = \frac{2}{3} t$ for wrought iron down to $u = \frac{1}{2} t$ for tool steel.

Second.—With a stress alternating from a positive amount, $+ v$, to an equal negative amount, $- v$, the bar will ultimately break if $v =$ about $\frac{1}{2} u$.

Third.—With a certain initial stress or Fixed Load (Min. S), and a certain Moving Load added and removed (causing the total stress to vary by loading and unloading from Min. S to Max. S), the stress on the bar becomes compound, being partly due to Fixed Load and partly due to Moving Load. Under these conditions, the total stress a under which the bar ultimately breaks will vary with the ratio or percentage of Fixed Load to Moving Load.

The extreme limits are: on one side "All Fixed Load," when

Range, Min. S to Max. $S = 0$, then $a = t$ and $\phi = 1$;

and on the other side, "All Moving Load," when

Range, Min. S to Max. $S = u$, then $a = u$ and $\phi = 0$.

Between these limits the value of a for any ratio of Fixed Load to Moving Load can be determined by means of "Gerber's parabola," or by the use of Launhardt's formula.*

Gerber's Parabola.—The range of stress which a bar will bear for an indefinite number of repetitions of the load applied and removed having been determined by Wöhler's experiments for different conditions of loading, it was found by Gerber that if the ranges of stress be plotted as ordinates, and the corresponding minimum stresses as abscissas, the points would fall approximately on a parabolic curve.

Let $\triangle =$ The range of stress, *i. e.*, Max. $S \mp$ Min. S.†

$t =$ The greatest stress the bar will bear under a static load.

$k =$ A constant for the material.

Then Gerber's equation may be written thus—

$$(\text{Min. } S + \tfrac{1}{2} \triangle)^2 + k \triangle = t^2$$

The ultimate static breaking stress for wrought iron, as ordinarily used for bridge girders, may be taken as 21 tons per square inch. For iron of this quality, the results of Wöhler's experiments show that the

* For an account of other methods of dealing with the subject, which have been proposed by Schäffer, Müller, Winkler, Cain, Smith, Seefehlner, Ritter, Lippold, and Clericetti, with tabular comparisons of results, see the paper by Weyrauch published in *Proc. Inst. C. E.*—1882-83.—Vol. lxxi, p. 298.

† The upper sign is to be taken where the stresses are of the same kind and the lower, if of different kinds (*i. e.*, ranging between tension and compression).

greatest stress of which the bar will bear an indefinite number of repetitions with the load applied and removed, will be about 14 tons, *i. e.*, about two-thirds of the ultimate static breaking stress. The corresponding value of k in Gerber's equation will then be 28.

The general results for cumulative effect of the Moving Load, obtained by the use of Gerber's Parabola for wrought iron, are given in Table No. 2, for different conditions of loading from "All Moving Load," when :

Range from Min. S to Max. $S = u$, and $a = u$;

advancing by 5% differences up to "All Fixed Load," when,

Range from Min. S to Max. $S = 0$, and $a = t$.

TABLE No. 2.—Cumulative Effect of Moving Load Deduced from Wöhler's Experiments by Gerber's Parabola.

Nature of Compound Load. Ratio Percentage.		Cumulative effect of Moving Load compared with that of Fixed Load. Percentage.	Ultimate Breaking Stress for the Compound Load. Tons per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	150.00	14.00	0.00	14.00
2.5	97.5	149.00	14.21	0.36	13.86
5	95	148.00	14.42	0.72	13.70
10	90	146.00	14.85	1.49	13.37
15	85	144.00	15.28	2.29	12.99
20	80	142.01	15.72	3.14	12.57
25	75	140.05	16.15	4.04	12.11
30	70	138.12	16.58	4.97	11.60
33.3	66.7	136.85	16.86	5.62	11.24
35	65	136.22	17.00	5.95	11.05
40	60	134.37	17.41	6.96	10.45
45	55	132.57	17.81	8.01	9.80
50	50	130.81	18.20	9.10	9.10
55	45	129.12	18.57	10.21	8.36
60	40	127.48	18.92	11.35	7.57
65	35	125.91	19.25	12.52	6.74
66.7	33.3	125.40	19.36	12.91	6.45
70	30	124.40	19.57	13.70	5.87
75	25	122.96	19.86	14.90	4.97
80	20	121.58	20.13	16.11	4.03
85	15	120.26	20.38	17.32	3.06
90	10	119.00	20.61	18.55	2.06
95	5	117.81	20.81	19.77	1.04
100	0	116.67	21.00	21.00	0.00

Launhardt's Formula.—The formula proposed by Launhardt to express the results claimed by Wöhler's experiments is as follows:

$$a = u\left(1 + \frac{t-u}{u} \cdot \frac{\text{Min. } S}{\text{Max. } S}\right)$$

It will be observed that if the results obtained by the use of Launhardt's formula be plotted on the same system as that adopted by Gerber, the curve obtained will also be a parabola.

Taking the data for wrought iron as before (see under Gerber's parabola) there results:

For an entirely static load (*i. e.*, All Fixed Load) $t = 21$;

For a load alternating from zero to maximum (*i. e.*, All Moving Load) $u = 14$.

Let $\frac{\text{Min. } S}{\text{Max. } S}$ be represented by the symbol ϕ; then, for wrought iron, Launhardt's formula may be written thus:

$$a = 14 \left(1 + \frac{\phi}{2}\right)$$

The general results for the cumulative effect of the Moving Load, obtained by the use of Launhardt's formula for wrought iron, are given in Table No. 3 for different conditions of loading from "All Moving Load," when:

Range from Min. S to Max. $S = u$, then $a = u$ and $\phi = 0$,

advancing by 5% differences up to "All Fixed Load," when:

Range from Min. S to Max. $S = 0$, then $a = t$ and $\phi = 1$.

Total Effect of the Moving Load.

Definition of Total Effect.—The total extra effect produced by the Moving Load, as compared with that due to the same weight as Fixed Load, may be taken as that which would be produced by an indefinite number of repetitions of the immediate effect. In other words, the total effect which the bridge should be designed to bear with safety is the ultimate cumulative effect of the immediate effect.

Method of Calculation.—Let it be assumed, for example, that a certain member of a bridge girder has to be designed to bear a compound load, of which 10% is due to Fixed Load, and 90% due to Moving Load, giving a ratio of—

$$\frac{10 \text{ fixed}}{90 \text{ moving}}$$

and that the actual amount of the Moving Load is 100 tons.

This 100 tons of Moving Load is, owing to the violence with which it is applied, found by experiment (see Table No. 1) to produce an immediate effect as severe as that which would be produced by 122.63 tons applied quietly.

TABLE No. 3.—Cumulative Effect of Moving Load. Deduced from Wöhler's Experiments by Launhardt's Formula.

Nature of Compound Load. Ratio Percentage.		Cumulative effect of Moving Load compared with that of Fixed Load. Percentage.	Ultimate Breaking Stress for the Compound Load. Tons per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	150.00	14.00	0.00	14.00
2.5	97.5	149.38	14.18	0.35	13.82
5	95	148.78	14.35	0.72	13.63
10	90	147.62	14.70	1.47	13.23
15	85	146.51	15.05	2.26	12.79
20	80	145.45	15.40	3.08	12.32
25	75	144.44	15.75	3.94	11.81
30	70	143.48	16.10	4.83	11.27
33.3	66.7	142.86	16.33	5.44	10.89
35	65	142.55	16.45	5.76	10.69
40	60	141.67	16.80	6.72	10.08
45	55	140.82	17.15	7.72	9.43
50	50	140.00	17.50	8.75	8.75
55	45	139.22	17.85	9.82	8.03
60	40	138.46	18.20	10.92	7.28
65	35	137.74	18.55	12.06	6.49
66.7	33.3	137.50	18.67	12.44	6.22
70	30	137.04	18.90	13.23	5.67
75	25	136.36	19.25	14.44	4.81
80	20	135.71	19.60	15.68	3.92
85	15	135.09	19.95	16.96	2.99
90	10	134.48	20.30	18.27	2.03
95	5	133.90	20.65	19.62	1.03
100	0	133.33	21.00	21.00	0.00

Having found the enhancement of stress due to the violence with which the load is applied, the cumulative effect will evidently be that due to an indefinite number of repetitions of this enhanced stress. For cumulative effect, therefore, the load which is applied to and removed from the member every time the train traverses the bridge must be taken as 122.63 tons (instead of 100 tons). The Fixed Load, however, remains as before.

Hence, for the purpose of determining the cumulative effect, the ratio—

$$\frac{10 \text{ fixed}}{90 \text{ moving}}$$

will be changed, the Moving Load being increased by 22.63%, and the ratio will become—

$$\frac{10 \text{ fixed}}{110.367 \text{ moving}}$$

or as a percentage ratio—

$$\frac{8.308 \text{ fixed}}{91.692 \text{ moving}}$$

The coefficient for ultimate cumulative effect, applicable to this ratio by Gerber's Parabola, is 1.467. This coefficient is to be applied to 122.63 tons (not 100 tons) and—

$$122.63 \times 1.467 = 179.9 \text{ tons.}$$

By Launhardt's formula the results would be:

$$122.63 \times 1.480 = 181.5 \text{ tons.}$$

Mode of Application—Hence with a ratio of $\frac{10 \text{ fixed}}{90 \text{ moving}}$, a Moving Load of 100 tons applied and removed an indefinite number of times would produce a total effect (the cumulative effect of the immediate effect) equivalent to about 180 tons as fixed load. In other words, with this ratio, to obtain the equivalent of the Moving Load in terms of Fixed Load, the coefficient would be 1.8; and, to determine the dimensions of the member, the Moving Load may be multiplied by 1.8, the result added to the actual Fixed Load, and, for the purposes of calculation, the total so obtained used as "All Fixed Load."

Statement of Results.—The general results for the total effect of the Moving Load (*i. e.*, the ultimate cumulative effect of the immediate effect) for wrought iron are given in Table No. 4 for different combinations of loading, from "All Moving Load," advancing by 5% differences, up to "All Fixed Load." The total effect here given is calculated as explained above, the cumulative effect as used for calculation being taken as the mean between the results obtained by the use of Gerber's Parabola and those obtained by the use of Launhardt's formula. These results are also exhibited in a graphic form in Fig. 1.

Results Applied to Practice.

Results by Experiment.—For wrought iron, the actual effects for a Compound Load, as determined by experiment, for different combinations of Fixed Load and Moving Load, are shown in Table No. 4.

It will be remembered that these figures represent the combined results for immediate effect and for cumulative effect, and that for immediate effect the curve was not plotted by any rule, but was merely

TABLE No. 4.—Total Effect of Moving Load. The Ultimate Cumulative Effect of the Immediate Effect.

Nature of Compound Load. Ratio Percentage.		Total effect of Moving Load compared with that of Fixed Load. Percentage.	Ultimate Breaking Stress for the Compound Load. Tons Per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	.....	.....	.....	.....
2.5	97.5	220.31	9.66	0.24	9.42
5	95	199.04	10.82	0.54	10.28
10	90	180.68	12.17	1.22	10.96
15	85	170.42	13.14	1.97	11.17
20	80	163.16	13.95	2.79	11.16
25	75	157.44	14.68	3.67	11.01
30	70	152.75	15.34	4.60	10.74
33.3	66.7	150.30	15.73	5.24	10.48
35	65	149.07	15.92	5.57	10.35
40	60	145.98	16.46	6.59	9.88
45	55	143.31	16.96	7.63	9.33
50	50	140.90	17.43	8.72	8.72
55	45	138.70	17.89	9.84	8.05
60	40	136.63	18.32	10.99	7.33
65	35	134.71	18.73	12.17	6.55
66.7	33.3	134.13	18.85	12.57	6.29
70	30	132.94	19.11	13.38	5.73
75	25	131.28	19.48	14.61	4.87
80	20	129.73	19.82	15.86	3.96
85	15	128.36	20.14	17.12	3.02
90	10	127.13	20.45	18.40	2.04
95	5	126.02	20.73	19.69	1.04
100	0	125.00	21.00	21.00	0.00

drawn in evenly, to average, as nearly as practicable, the actual values obtained by experiment. The corresponding curve has, therefore, no equation, and the results can only be utilized by a reference to the table itself.

This process would, however, not be specially troublesome, as, in practice, a corresponding table or diagram would be used to facilitate computations made on any modern system, or based on an ordinary formula, such as that of Launhardt. It would, in either case, merely be necessary to ascertain the percentage ratios of Moving Load to Fixed Load, and take the corresponding figures from the table or diagram.

It is, nevertheless, certainly desirable to have a simple rule or easily remembered formula, and the following are accordingly offered for consideration, as giving results sufficiently near to those obtained by experiment, and as being, at the same time, easy of application. The effect of each of these systems, for wrought iron, is exhibited graph-

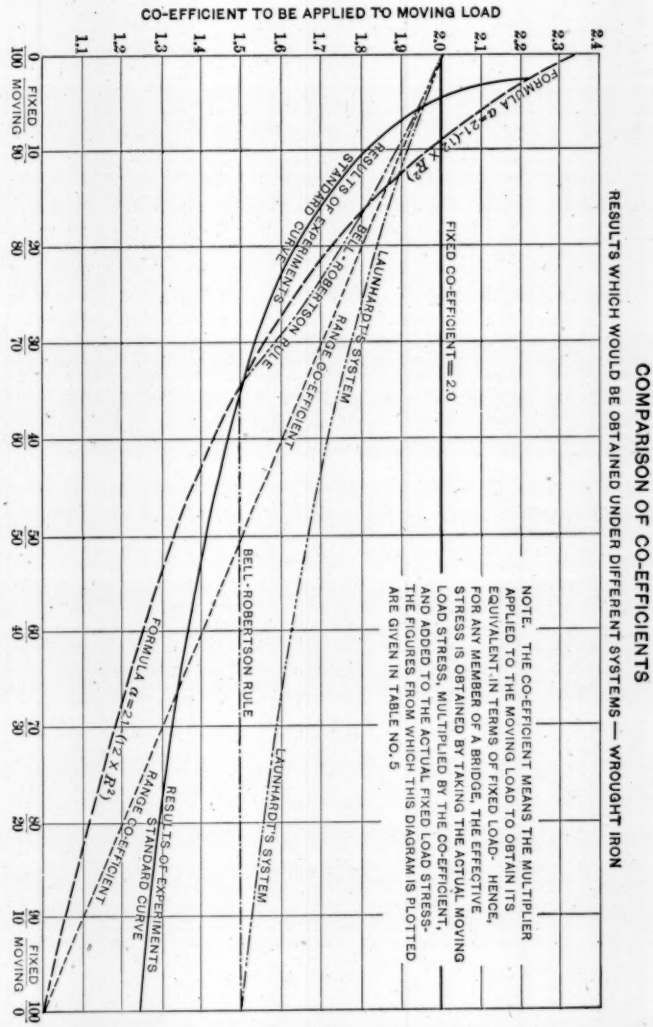
COMPARISON OF CO-EFFICIENTS
RESULTS WHICH WOULD BE OBTAINED UNDER DIFFERENT SYSTEMS — WROUGHT IRON
NOTE. THE CO-EFFICIENT MEANS THE MULTIPLIER APPLIED TO THE MOVING LOAD TO OBTAIN ITS EQUIVALENT IN TERMS OF FIXED LOAD- HENCE, FOR ANY MEMBER OF A BRIDGE, THE EFFECTIVE STRESS IS OBTAINED BY TAKING THE ACTUAL MOVING LOAD STRESS, MULTIPLIED BY THE CO-EFFICIENT, AND ADDED TO THE ACTUAL FIXED LOAD STRESS- THE FIGURES FROM WHICH THIS DIAGRAM IS PLOTTED ARE GIVEN IN TABLE NO. 5
CO-EFFICIENT TO BE APPLIED TO MOVING LOAD
FIXED CO-EFFICIENT = 2.0
FORMULA a=21-(12 X R²)
RESULTS OF EXPERIMENTS
STANDARD CURVE
BELL-ROBERTSON RULE
LAUNHARDT'S SYSTEM
RANGE CO-EFFICIENT
RATIO OF FIXED LOAD TO MOVING LOAD
FIXED
MOVING

Fig. 2.

ically in Figs. 2 and 3, for comparison with one another and with the results of experiment. The curve representing the results of experiment is, for distinction, called the "Standard Curve."

General Principles.—It is assumed as established, that, for any Compound Load, the variation in the breaking stress per unit of area depends on the range of stress; that is to say, on the relative amount of Moving Load in the Compound Total Load. To assist in the investigation of this subject, and to enable different systems to be properly compared, it will, therefore, be convenient to adopt a symbol which will directly represent the range. For stresses all in the same direction (*i. e.*, all in tension or all in compression): Range of stress = Moving Load = Max. S — Min. S.

$$\text{Let } R = \frac{\text{Moving Load}}{\text{Moving Load} + \text{Fixed Load}} = \frac{\text{Range}}{\text{Total}}$$

Then the value of R increases directly as the range of stress, and represents the ratio which the range of stress bears to the total stress.

The extreme limits in the value of R are, on one side, "All Fixed Load," when:

$$\text{Min. } S = \text{Max. } S, \text{ then } a = t \text{ and } R = 0,$$

and on the other side, "All Moving Load" when:

$$\text{Min. } S = 0, \text{ then } a = u \text{ and } R = 1.$$

It will be observed that R is thus the complement of the symbol used for the Launhardt formula, thus:—

$$R + \phi = 1,\ R = 1 - \phi,\ \phi = 1 - R$$

and, remembering this relation, it is easy to translate an expression from one system to the other.

Fixed Coefficient.—Under this system each ton of Moving Load is assumed to have a constant relative value, as compared with a ton of Fixed Load, and, on any member of a bridge, the nominal unit stress would merely depend upon how much Moving Load and how much Fixed Load had to be compounded at that relative value.

It will be seen, however, from the results of experiment, that the effect of Moving Load, as compared with that of Fixed Load, is not in all cases the same, but is relatively greater as the proportion of Moving Load becomes higher. Thus, according to the results of experi-

ment, the coefficients proper to different conditions of loading are as follows:

Moving Load.	Fixed Load.	Actual Coefficient.*
10	90	1.27
50	50	1.41
90	10	1.81

It might, therefore, at first sight appear that under the single coefficient system it would not be possible to obtain a graduated scale for the permissible unit stress which should be fairly in accord with the requirements of the case; but it will be found on consideration that under this system the graduation is better than might at first be supposed; and that a single coefficient, if selected as applicable to a high ratio of Moving Load to Fixed Load, would in itself provide a sliding scale yielding results which would perhaps nowhere differ to an important extent from those which would be obtained under a more complete system.

Remembering that the coefficient is applied only to that part of the Total Load which has been called "Moving Load," it is evident that its effect in increasing the size of any member becomes relatively small as the proportion of "Moving Load" to be taken by that member is reduced. Thus, with a single coefficient selected as applicable to a high ratio of Moving Load as compared with Fixed Load, the percentage of error in the Total Load obtained by its use would be decreased roughly in proportion as the actual error in the coefficient itself became greater; and with the maximum error in the coefficient, as in the condition of (practically) all Fixed Load, the error in the Total Load used for the determination of the dimensions of a member would become (practically) nil.

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* As there has been some misunderstanding with regard to this coefficient, the following illustration is offered with an apology to those who, being well acquainted with the matter, will no doubt regard it as superfluous:

Suppose that a number of bullets, all of the same size, be made, some of copper and some of gold; and that each copper bullet weighs one ounce.

Suppose further, in the first instance, that the specific gravity of gold be exactly double that of copper. Under these conditions, with any mixed lot of bullets—some of copper and some of gold—it is evident that the weight of the lot in ounces could at once be ascertained by counting the number of each kind separately, multiplying the gold number by two, and adding the result to the actual copper number. This represents the case for a single fixed coefficient.

Suppose now the conditions altered, and that the specific gravity of the gold bullets does not remain the same, but in a mixed lot is found to be higher when the gold bullets are relatively numerous, and lower when the reverse conditions prevail. To obtain the total weight for any mixed lot now, not only would the number of bullets of each metal have to be counted, but the ratio of the numbers would have to be determined, and the calculation be based on a specific gravity for gold which would vary with that ratio. This represents the case for a variable coefficient.

This will be apparent from the following example:

Case 1.—Conditions assumed: Moving Load, 9 tons; Fixed Load, 1 ton; Correct Coefficient, 2.0.

Here the correct equivalent Total Load would be $18 + 1 = 19$ tons. Had the coefficient been wrongly taken at 1.5, the equivalent Total Load would have worked out to $13.5 + 1 = 14.5$ tons, showing an error in the result nearly proportioned to the error in the coefficient, and all on the wrong side.

Case 2.—Conditions assumed: Moving Load, 1 ton; Fixed Load, 9 tons; Correct Coefficient, 1.3.

Here the correct equivalent Total Load would be $1.3 + 9 = 10.3$ tons. Had the coefficient been wrongly taken at 2.0, the equivalent Total Load would have worked out to $2 + 9 = 11$ tons, showing an error in the result very small as compared with the error in the coefficient, and all on the side of safety.

This system, with a fixed coefficient of 2.0, has found much favor in America, and has been adopted by several bridge engineers, notably by Theodore Cooper, M. Am. Soc. C. E. It has the advantage of very great simplicity, and gives results which show a very fair approximation to those obtained by experiment.

The corresponding formula is:

$$\text{Safe working stress in tons per square inch} = 7 \times \left(\frac{1}{1 + R}\right)$$

It will be observed on reference to Fig. 3, that the greatest percentage divergence from the standard curve occurs at the point where $\frac{\text{Fixed Load} = 40}{\text{Moving Load} = 60}$; here the results with a fixed coefficient of 2.0 would give an excess of strength of about 25 per cent.

In Fig. 2, for the coefficient, this system is represented by a straight line parallel to the datum line. In Fig. 3, for safe working stress, the bend of the curve is downward toward the datum line throughout, whereas the bend of the standard curve is in the opposite direction, with curvature increasing with the ratio of Moving Load. This peculiarity causes a considerable difference in the direction of the two curves where the ratio of Moving Load is high. The curve here in fact bends the wrong way.

Rule Adopted in India.—It will be observed that, where a fixed coefficient of 2.0 is adopted for all members of a truss, there will be an excess of strength for those members in which the Total Load is made

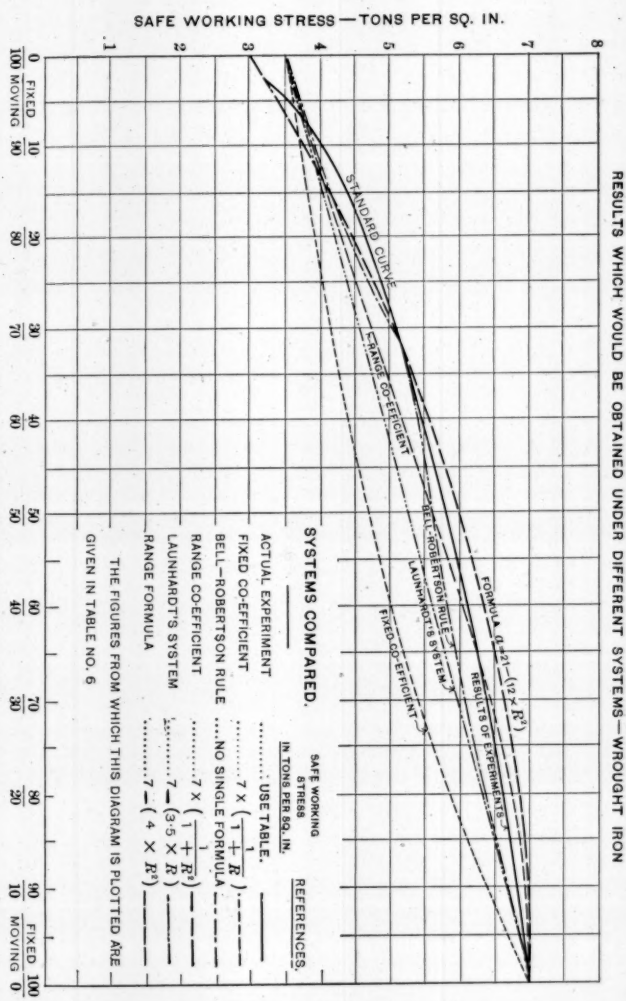
SAFE WORKING STRESS
RESULTS WHICH WOULD BE OBTAINED UNDER DIFFERENT SYSTEMS—WROUGHT IRON
SAFE WORKING STRESS—TONS PER SQ. IN.
STANDARD CURVE
RANGE CO-EFFICIENT
BELL-ROBERTSON RULE
LAUNHARDT'S SYSTEM
FIXED CO-EFFICIENT
FORMULA $a = 21 - (12 \times R^2)$
RESULTS OF EXPERIMENTS
SYSTEMS COMPARED.
SAFE WORKING STRESS IN TONS PER SQ. IN.
REFERENCES.
ACTUAL EXPERIMENT ..... USE TABLE.
FIXED CO-EFFICIENT ..... $7 \times \left(\frac{1}{1+R}\right)$
BELL-ROBERTSON RULE ..... NO SINGLE FORMULA
RANGE CO-EFFICIENT ..... $7 \times \left(\frac{1}{1+R^2}\right)$
LAUNHARDT'S SYSTEM ..... $7 - (3{\cdot}5 \times R)$
RANGE FORMULA ..... $7 - (4 \times R^2)$
THE FIGURES FROM WHICH THIS DIAGRAM IS PLOTTED ARE GIVEN IN TABLE NO. 6
FIXED 0 10 20 30 40 50 60 70 80 90 100
MOVING 100 90 80 70 60 50 40 30 20 10 0
RATIO OF FIXED LOAD TO MOVING LOAD

FIG. 8.

up of a large proportion of Fixed Load. This will be specially observable in the main booms of bridges of large span, for which a coefficient of 1.5 would be more suitable.

With a view to retain the simplicity of the fixed coefficient system, and, at the same time, obtain results more nearly in accord with actual conditions, the following rule has been adopted by the Government of India:

For any member of a railway bridge of wrought iron or steel, the total working load is to be taken as the greatest "Moving Load" multiplied by a coefficient and added to the actual "Fixed Load."

The coefficient to be used for this purpose is 2.0 in all cases, except for the upper and lower booms of triangulated girders, for which a coefficient of 1.5 may be used.

It will be seen that, although two coefficients are used, there will be no step or kink in the line representing the rule graphically. In the graphic representation of the rule there would be two separate lines, one for main booms and the other for general work.

Under the operation of the rule the coefficient of 1.5 would not be used for very small girders, inasmuch as in modern practice in India triangulated girders are not used for spans of less than 80 ft.

It has been shown that, with only a single coefficient, the adjustment of the unit stress, to suit varying ratios of Fixed Load to Moving Load, can be arranged for fairly well, provided the coefficient selected be that applicable to a condition of approximately "All Moving Load." With the addition of the second coefficient for main booms, the provisional rule adopted in India gives results which are probably as nearly in accord with the actual conditions as could be attained without the use of a variable coefficient. The rule has, moreover, the practical advantage of being simple and very easily applied.

This rule is practically in accord with that advocated by T. Claxton Fidler in his able and interesting exposition of his "Dynamic Method."* The formula given by Fidler is

$$\Omega = \text{Max. } S + \omega$$

where Ω represents the momentary internal stress and ω the dynamic increment. For cross-girders, vertical suspenders, diagonals of web bracing and girders up to a span of 20 ft.

$$\omega = \text{Max. } S - \text{Min. } S;$$

* "A Practical Treatise on Bridge Construction," by T. Claxton Fidler, M. Inst. C. E. Edition 1893; p. 260.

this implies that the "effective stress" due to the Moving Load will be obtained by adding a "dynamic increment" equal to the Moving Load. In other words, the Moving Load is to be doubled to obtain its equivalent in terms of Fixed Load.

For main booms of girders of 100 ft. span or upward

$$\omega = \frac{\text{Max. } S - \text{Min. } S}{2};$$

this implies that the "effective stress" due to the Moving Load will be obtained by adding a "dynamic increment" equal to half the Moving Load. In other words, the Moving Load is to be increased by half as much again to obtain its equivalent in terms of Fixed Load.

Bell-Robertson Rule.—This rule was proposed by Messrs. J. R. Bell and F. E. Robertson as an improvement to the Government of India rule given above; the object being to obtain a simple means of varying the coefficient according to the nature of the load.

The rule is as follows:

For any member of a railway bridge of wrought iron or steel, the total "working load" is to be taken as the greatest "live load" multiplied by a coefficient of 1.5 and added to the actual "dead load," provided that a minimum dead load equal to half the live load shall be taken into calculation wherever the actual dead load is less.

Hence, under this rule, the coefficient would be 2.0 for the condition of All Moving Load, and would decrease as the relative amount of Moving Load became less, until at the point $\frac{\text{Moving Load} = 66.7}{\text{Fixed Load} = 33.3}$, the coefficient would become 1.5. From this point onward there would be a fixed coefficient of 1.5.

This rule gives a good approximation to the results as obtained by experiment, and is at the same time simple and easy of application. It is, however, not a uniformly consistent rule, and, if represented graphically, will be seen to be made up of two curves of different characters, with their intersections at the point $\frac{\text{Moving Load} = 66.7}{\text{Fixed Load} = 33.3}$. There is thus a cusp or kink in the line at this point. (See Fig. 2.)

Range Coefficient.—With a coefficient increasing with the range of stress (directly as $1 + R$), a very good approximation is obtained. On this system it is assumed that the extra effect of the Moving Load, as compared with the same weight as Fixed Load, may, for practical pur-

poses, be taken as the nominal Moving Load multiplied by R. The coefficient to be applied to the Moving Load, to obtain its equivalent in terms of Fixed Load, will then be $1 + R$.*

The practical application of this rule is extremely simple, as the coefficient for any compound load is obtained at once by merely adding 1.0 to the decimal giving the proportion of Moving Load to Total Load.

Example:

Moving Load 0.9 of Total Load.

Then coefficient = 1.9.

Moving Load 0.66 of Total Load.

Then coefficient = 1.66.

The corresponding formula is:

$$\text{Safe working stress in tons per square inch} = 7 \times \left(\frac{1}{1+R^2}\right)$$

In Fig. 2, for the coefficient, this system is represented by a straight line drawn from the point 2.0 for "All Moving Load," to 1.0 for "All Fixed Load."

In Fig. 3, for safe working stress, the curve representing this system has a double (or reversed) curvature. It will be seen that it gives a very good approximation to the standard curve throughout.

Launhardt's System.—Launhardt's formula is:

$$a = u\left(1 + \frac{t-u}{u} \cdot \frac{\text{Min. } S}{\text{Max. } S}\right)$$

Avoiding troublesome fractions, the nearest approximation to the results for total effect, obtained by experiment, will be found by assigning to u a value of one half t. For wrought iron, t being taken as 21 tons, the formula would then become—

$$\text{Breaking stress in tons per square inch} = 10.5 \times (1 + \phi);$$

and with a factor of safety of 3.0—

$$\text{Safe working stress in tons per square inch} = 3.5 \times (1 + \phi)$$

It is to be noted that this result is the same as that which has been arrived at in America by an entirely independent course of reasoning, and is now adopted in some of the best modern American specifications.

Using the symbol R instead of ϕ, the formula would be written thus:

$$\text{Safe working stress} = 3.5\,(2 - R)$$
$$= 7.0 - 3.5\,R.$$

* As a peculiarity incidental to this system, it may be noted that for any Compound Load in which Fixed Load + Moving Load = 100, the equivalent (or effective) Total Load will be $100 + (10\,R)^2$. For example, $\frac{\text{Fixed Load} = 20}{\text{Moving Load} = 80}$, then $R = 0.8$, and the effective Total Load is 164.

The use of this formula, with the values adopted, involves the following assumptions:

(*a*) The coefficient to be applied to the Moving Load, to obtain its equivalent in terms of Fixed Load, is 2.0 for the condition of "All Moving Load." diminishing as the ratio of Moving Load to Fixed Load decreases, until it becomes 1.5 for the condition of "All Fixed Load." The variation in the value of the coefficient is directly in the ratio of Total Load to Fixed plus Total Load. (See Fig. 2.)

(*b*) The nominal breaking stress varies directly as the ratio of Fixed Load to Total Load. Hence, in Fig. 3, for "nominal" safe working stress, this system is represented by a straight line.

This formula is very simple in application and has the advantage of being already well known and established in practice in America. It will also be seen by a reference to Fig. 3 that the results obtained on this system give a fairly good approximation to the results obtained by experiment.

It will be observed, however, that, being a straight line formula (see Fig. 3), it is impossible by any adjustment of the relative values of u and t to obtain results for safe working stress decreasing in a higher ratio as the percentage of Moving Load becomes greater. To give such results, and thereby be more in accord with the facts as indicated by experiment, the line as plotted in Fig. 3 should bend upward, *i. e.*, be convex as viewed from the top of the diagram.

Range Formula.—It being established that the breaking stress for a bar becomes less as the range of stress increases, it is evident that a useful formula may be constructed under which the breaking stress for a compound load may be arrived at by means of an expression which will show directly, as a function of the range, the amount by which the static breaking stress is to be reduced. On the system now proposed, therefore, the static breaking stress will be the standard for comparison in all cases, and the lower breaking stress for any compound load will be ascertained by a direct subtraction of the amount by which the breaking stress is reduced.

Avoiding troublesome fractions and complicated expressions, it is considered that for wrought iron, with a static breaking stress of 21 tons per square inch, the most suitable formula for general use would be:

Breaking stress, in tons per square inch, $= 21 - (12 \times R^2)$; and with a factor of safety of 3:

Safe working stress, in tons per square inch, $= 7 - (4 \times R^2)$

In Fig. 2, for the coefficient, the results obtained by the use of this formula are represented by a line extending from 2.33 for "All Moving Load," to 1.00 for "All Fixed Load."

In Fig. 3, for safe working stress, it will be observed that for the lower ratios of Moving Load, the working stress which would be allowed under this formula is somewhat higher than that given by the standard curve. The greatest percentage of difference is at the point where the Moving Load is about half the Fixed Load — here, under the formula, the permissible working stress per square inch would be 6.56 tons, as against 6.28 tons indicated as suitable by the results of experiment. The line represented by this formula crosses the standard curve near the point where the Moving Load is about double the Fixed Load; and thence for the higher ratios of Moving Load the working stress which would be allowed under the formula is lower than that indicated by the experimental results. At the point where the compound load becomes "All Moving Load," the permissible working stress, as given by the formula, is 3.0 tons per square inch.

General Summary.—The six systems, which have now been compared and discussed, for giving approximately the safe working stress for wrought iron, are:*

(*a*) *Actual Experimental Results:*

Safe working stress obtained by use of table.

(*b*) *Fixed Coefficient:*

Safe working stress in tons per square inch $= 7 \times \left(\frac{1}{1+R}\right)$

(*c*) *Bell-Robertson Rule:*

No single formula applicable.

(*d*) *Range Coefficient:*

Safe working stress in tons per square inch $= 7 \times \left(\frac{1}{1+R^3}\right)$

(*e*) *Launhardt's System:*

Safe working stress in tons per square inch $= 7 - (3.5 \times R)$

(*f*) *Range Formula:*

Safe working stress in tons per square inch $= 7 - (4 \times R^2)$

---

* In these formulas the symbol R represents the proportionate range of stress, thus:

$$R = \frac{\text{Moving Load}}{\text{Fixed Load} + \text{Moving Load}} = \frac{\text{Range}}{\text{Total}}$$

The "Safe Working Stress" in each case is "Nominal Stress" due to the weight of the Moving Load and Fixed Load simply added. The corresponding "Effective Stress" obtained by the use of these formulas is 7 tons per square inch throughout.

TABLE No. 5.—Results for the Coefficient. Wrought Iron.

Nature of Compound Load. Ratio Percentage.		Effect of Moving Load—Percentage. Compared with the Effect Due to the Same Weight Applied as Fixed Load.					
		Results of Experiment for Actual Total Effect.	Fixed Coefficient. 2.0	Bell-Robertson Rule.	Range Coefficient.	Launhardt's System $u = \frac{1}{2}t$.	Range Formula.
Fixed Load.	Moving Load.		$a = 21 \times \left(\frac{1}{1+R}\right)$		$a = 21 \times \left(\frac{1}{1+R^2}\right)$	$a = 21 - \left(10.5 \times R\right)$	$a = 21 - \left(12 \times R^2\right)$
0	100	.....	200.00	200.00	200.00	200.00	233.33
2.5	97.5	230.31	200.00	197.44	197.50	197.56	221.96
5	95	199.04	200.00	194.74	195.00	195.24	212.09
10	90	180.68	200.00	188.89	190.00	190.91	195.74
15	85	170.42	200.00	182.35	185.00	186.96	182.73
20	80	163.16	200.00	175.00	180.00	183.33	172.07
25	75	157.44	200.00	166.67	175.00	180.00	163.16
30	70	152.75	200.00	157.14	170.00	176.92	155.56
33.3	66.7	150.30	200.00	150.00	166.67	175.00	151.06
35	65	149.07	200.00	150.00	165.00	174.07	148.96
40	60	145.93	200.00	150.00	160.00	171.43	143.17
45	55	143.31	200.00	150.00	155.00	168.96	138.00
50	50	140.90	200.00	150.00	150.00	166.67	133.33
55	45	138.70	200.00	150.00	145.00	164.51	129.08
60	40	136.63	200.00	150.00	140.00	162.50	125.16
65	35	134.71	200.00	150.00	135.00	160.60	121.51
66.7	33.3	134.13	200.00	150.00	133.33	160.00	120.34
70	30	132.94	200.00	150.00	130.00	158.82	118.07
75	25	131.28	200.00	150.00	125.00	157.14	114.81
80	20	129.73	200.00	150.00	120.00	155.56	111.70
85	15	128.36	200.00	150.00	115.00	154.05	108.68
90	10	127.13	200.00	150.00	110.00	152.63	105.75
95	5	126.02	200.00	150.00	105.00	151.27	102.86
100	0	125.00	200.00	150.00	100.00	150.00	100.00

It will be observed that, of these systems, each has some advantage to recommend it, and that in the selection of the most suitable system for general use, it is to some extent a question of how far it may be considered advisable to sacrifice simplicity to obtain a closer approximation to the results obtained by experiment.

For convenience of comparison the results which would be obtained by the use of each rule or formula for different ratios of Fixed Load to Moving Load are exhibited in the following tables:

Table No. 5.—Results for the Coefficient.

Table No. 6.—Results for Safe Working Stress.

Table No. 7.—Percentage Comparison.

From the latter table can be ascertained at a glance the practical effect of any rule or formula on the area in cross-section of a member as compared with the area which would be indicated as the result of experiment.

TABLE No. 6.—Results for Safe Working Stress. Wrought Iron.

Nature of Compound Load. Ratio Percentage.		Safe Working Stress with Factor of Safety = 3.0. Tons per Square Inch.					
Fixed Load.	Moving Load.	Results of Experiment for Actual Total Effect.	Fixed Coefficient 2.0 $a = 21 \times \left(\frac{1}{1+R}\right)$	Bell-Robertson Rule.	Range Coefficient. $a = 21 \times \left(\frac{1}{1+R^2}\right)$	Launhardt's System. $u = \frac{1}{2}t$ $a = 21 - (10.5 \times R)$	Range Formula. $a = 21 - (12 \times R^2)$
0	100	.....	3.50	3.50	3.50	3.50	3.00
2.5	97.5	3.22	3.54	3.59	3.59	3.59	3.20
5	95	3.61	3.59	3.68	3.68	3.67	3.39
10	90	4.06	3.68	3.89	3.87	3.85	3.76
15	85	4.38	3.78	4.12	4.06	4.02	4.11
20	80	4.65	3.89	4.38	4.27	4.20	4.44
25	75	4.89	4.00	4.67	4.48	4.37	4.75
30	70	5.11	4.12	5.00	4.70	4.55	5.04
33.3	66.7	5.24	4.20	5.25	4.85	4.67	5.22
35	65	5.31	4.24	5.28	4.92	4.72	5.31
40	60	5.49	4.37	5.38	5.15	4.90	5.56
45	55	5.65	4.52	5.49	5.37	5.07	5.79
50	50	5.81	4.67	5.60	5.60	5.25	6.00
55	45	5.96	4.83	5.71	5.82	5.42	6.19
60	40	6.11	5.00	5.83	6.03	5.60	6.36
65	35	6.24	5.18	5.96	6.24	5.77	6.51
66.7	33.3	6.28	5.25	6.00	6.30	5.83	6.56
70	30	6.37	5.38	6.09	6.42	5.95	6.64
75	25	6.49	5.60	6.22	6.59	6.12	6.75
80	20	6.61	5.83	6.36	6.73	6.30	6.84
85	15	6.71	6.09	6.51	6.85	6.47	6.91
90	10	6.82	6.36	6.67	6.93	6.65	6.96
95	5	6.91	6.67	6.83	6.98	6.82	6.99
100	0	7.00	7.00	7.00	7.00	7.00	7.00

The results are also exhibited graphically for the coefficient in Fig. 2, and for safe working stress in Fig. 3.

With reference to the remarks made on this subject when discussing the fixed coefficient system, it will be observed that where the ratio of Fixed Load, as compared with Moving Load, is high, very great differences in the coefficient, as given in Table No. 5, have but a small effect on the practical results, as given in Tables Nos. 6 and 7.

Approximate Nature of Results.—It is to be remembered that the figures here quoted as "determined by experiment" are merely the averages of a large number of results, among which there are great discrepancies; and that for each point determined the maximum and minimum often differ widely from each other. Further, that in deciding where to place the average value, various circumstances must be taken into account and allowed due weight; but the precise amount of correction required must in each case (within certain limits) remain

TABLE No. 7.—PERCENTAGE COMPARISON. WROUGHT IRON.

Nature of Compound Load. Ratio Percentage.		Area of Members in Cross-Section. Compared with the Area Indicated by Experiment. Percentage.					
Fixed Load.	Moving Load.	Results of Experiment for Actual Total Effect.	Fixed Coefficient 2.0 $a = 21 \times \left(\frac{1}{1+R}\right)$	Bell-Robertson Rule.	Range Coefficient. $n = 21 \times \left(\frac{1}{1+R^2}\right)$	Launhardt's System. $u = \frac{1}{2}t$ $a = 21 - (10.5 \times R)$	Range Formula. $a = 21 - (12 \times R^2)$
0	100						
2.5	97.5	100	90.89	89.74	89.76	89.79	100.74
5	95	100	100.48	97.89	98.03	98.14	106.39
10	90	100	110.08	104.28	104.86	105.33	107.85
15	85	100	115.74	106.35	107.74	108.79	106.54
20	80	100	119.57	106.29	108.94	110.72	104.74
25	75	100	122.31	104.84	109.21	111.83	103.00
30	70	100	124.16	102.25	108.83	112.36	101.43
33.3	66.7	100	124.81	99.87	108.17	112.33	100.38
35	65	100	125.09	100.46	107.85	112.32	99.95
40	60	100	125.43	101.91	106.62	111.99	98.70
45	55	100	125.17	102.97	105.19	111.40	97.64
50	50	100	124.53	103.78	103.78	110.70	96.86
55	45	100	123.49	104.33	102.42	109.89	96.31
60	40	100	122.11	104.66	101.17	109.02	96.00
65	35	100	120.38	104.77	100.09	108.08	95.88
66.7	33.3	100	119.71	104.78	99.76	107.74	95.87
70	30	100	118.31	104.66	99.20	107.07	95.94
75	25	100	115.93	104.34	98.54	106.00	96.18
80	20	100	113.26	103.83	98.17	104.87	96.60
85	15	100	110.30	103.11	98.08	103.70	97.17
90	10	100	107.09	102.23	98.33	102.48	97.92
95	5	100	103.65	101.18	98.96	101.25	98.86
100	0	100	100.00	100.00	100.00	100.00	100.00

a matter for judgment and discretion. In the interpretation of the results of Wöhler's experiments, for example, it will be seen from Tables Nos. 2 and 3 that such eminent authorities as Gerber and Launhardt are by no means in accord as to the value of the coefficient where the ratio of Fixed Load is high, the coefficient for wrought iron for the condition of "All Fixed Load," according to Launhardt, being one-seventh greater than that obtained by Gerber.

On this subject Weyrauch remarks as follows : *

"Considering that absolutely exact laws for constructive materials will certainly never result from experiments, that even in brands of iron acknowledged to be good, differences in the statical breaking strength t of as much as 40% occur, and that it is merely a question of finding a substitute for the still more rough and incorrect assumption of a constant a, even the preceding might suffice for practical purposes until more facts are accumulated.

* *Min. Proc. Inst. C. E.*, 1880-81, Vol. lxiii, pp. 282 and 283.

"Even for more exact determinations than those under consideration such an approximation ought to be considered satisfactory. To the author's mind it would appear sufficient if the deviations of the real values of a from those given by the formula did not exceed the deviations from one another of real values of a in good and commonly used materials."

If, therefore, by means of a simple rule or easily applied formula a fairly good approximation be obtained to the figures representing the result of experiment, it will evidently not be worth while to adopt a more complicated or troublesome system in order to secure a very close approximation to a line, the correct position of which (within certain limits) is after all a matter of some uncertainty.

The Selection of a System.—In the immediate effect, as determined by experiments, it is to be noted that the only effects allowed for are those which produce a measurable elongation or shortening of a bar or deflection of a beam, and the effect of the violent concussion or vibration, with which the application of the load is accompanied, is not taken into account.

It appears reasonable, however, to suppose that where the application of a load is accompanied by violent jarring or shock, there must be an effect on the bar more severe than that indicated by the mere temporary elongation, shortening or deflection.* The effect of jarring and vibration would be most severe with members (such as cross-girders and rail-bearers) which are exposed more immediately to the action of the locomotive, and generally with members for which the ratio of Moving Load is high as compared with Fixed Load. Hence, in making allowance for the effect of shocks and violent jarring and vibration, it will be proper to arrange that the allowance shall increase as the ratio of Moving Load to Total Load becomes greater, or, in other words, as the value of R becomes higher.

* In this connection the following illustrations are offered for consideration :—

Let two precisely similar bars be subjected to repeated deflections, equal in extent, involving stress beyond the elastic limit. Let the deflections of the first bar be produced by steady pressure applied and removed. Let the deflections (equal in extent) of the second bar be produced by successive blows of a hammer. It would be expected that the number of deflections before breaking would be less in the second case than in the first.

Again—Let a glass tube be supported in a vertical position, and closed at the lower end. Into this tube let a certain quantity of dry angular sand, not sufficient to fill the tube, be poured in quietly. Having marked on the tube the height occupied by the sand, let the sand be emptied out and again poured in as before. This second time, however, let the tube be violently jarred and shaken as the sand falls into it. In the second case, the sand will not stand at so great a height in the tube as in the first case.

Again—Let a bar of steel be placed with its axis pointing to the magnetic pole, and be left undisturbed. In the course of a few years it will have become to some extent magnetic. The same bar in that position if violently jarred by the blows of a hammer would have become equally magnetic in the course of a few minutes.

TABLE No. 8.—RESULTS FOR WROUGHT IRON. OBTAINED BY USE OF RANGE FORMULA. BREAKING STRESS = $21 - (12 \times R^2)$.

NATURE OF COMPOUND LOAD. RATIO PERCENTAGE.		BREAKING STRESS FOR THE COMPOUND (NOMINAL) LOAD. TONS PER SQUARE INCH.			Coefficient to obtain equivalent of Moving Load in terms of Fixed Load.	PERMISSIBLE WORKING STRESS.	
			Apportionment of the Breaking Stress.			Factor of Safety = 3.	
Fixed Load.	Moving Load.	For the Total Load.	Due to Fixed Load.	Due to Moving Load.		Nominal Stress. Tons per Square Inch.	Effective Stress. Tons per Square Inch.
0	100	9.0000	0.0000	9.0000	2.3333	3.0000	7.00
2.5	97.5	9.5928	0.2398	9.3530	2.2196	3.1976	7.00
5	95	10.1700	0.5085	9.6615	2.1209	3.3900	7.00
10	90	11.2800	1.1280	10.1520	1.9574	3.7600	7.00
15	85	12.3300	1.8495	10.4805	1.8273	4.1100	7.00
20	80	13.3200	2.6640	10.6560	1.7207	4.4400	7.00
25	75	14.2500	3.5625	10.6875	1.6316	4.7500	7.00
30	70	15.1200	4.5360	10.5840	1.5556	5.0400	7.00
33.3	66.7	15.6672	5.2224	10.4448	1.5106	5.2224	7.00
35	65	15.9300	5.5755	10.3545	1.4896	5.3100	7.00
40	60	16.8800	6.7520	10.0080	1.4317	5.5600	7.00
45	55	17.3700	7.8165	9.5535	1.3800	5.7900	7.00
50	50	18.0000	9.0000	9.0000	1.3333	6.0000	7.00
55	45	18.5700	10.2135	8.3565	1.2908	6.1900	7.00
60	40	19.0800	11.4480	7.6320	1.2516	6.3600	7.00
65	35	19.5300	12.6945	6.8355	1.2151	6.5100	7.00
66.7	33.3	19.6668	13.1112	6.5556	1.2034	6.5556	7.00
70	30	19.9200	13.9440	5.9760	1.1807	6.6400	7.00
75	25	20.2500	15.1875	5.0625	1.1481	6.7500	7.00
80	20	20.5200	16.4160	4.1040	1.1170	6.8400	7.00
85	15	20.7300	17.6205	3.1095	1.0868	6.9100	7.00
90	10	20.8800	18.7920	2.0880	1.0575	6.9600	7.00
95	5	20.9700	19.9215	1.0485	1.0286	6.9900	7.00
100	0	21.0000	21.0000	0.0000	1.0000	7.0000	7.00

On the other hand, of the irregular or abnormal effects to be covered by the factor of safety, it will be observed that many of these causes of extra unit stress would have but little effect on a large and massive member on which the ratio of Fixed Load is high as compared with Moving Load; and it would therefore appear proper to allow a somewhat higher effective stress* as the ratio of Moving Load to Total Load becomes less, or, in other words, as the value of R becomes lower.

Range Formula Recommended.—These results are attained for wrought iron by the use of the range formula:

Safe working stress in tons per square inch $= 7 - (4 \times R^2)$.

* Effective stress means the actual working stress to which the material is subjected, on the assumption that the coefficient adopted correctly represents the real effects of Moving Load as compared with that of the same weight as Fixed Load. In other words, "Effective Stress" means the stress due to the Fixed Load added to that due to the equivalent of the Moving Load in terms of Fixed Load.

With this formula it will be seen, by an inspection of Table No. 6 and of Fig. 3, that, as compared with the actual results of experiment, a somewhat higher effective stress would be allowed on the more massive parts of a large bridge truss where the irregular and abnormal effects would be least, and where economy of material may most profitably be exercised. On the other hand, with the lighter members of a truss (where the ratio of Moving Load is high as compared with Fixed Load, and where the irregular and abnormal effects would be most severely felt), the effective stress allowed would be somewhat lower than that deduced from the results of experiment.

The general results for wrought iron, obtained by the use of this formula, are exhibited in Table No. 8.

Results for Steel.

It will be observed that the value of u, as determined by Wöhler's experiments for steel, is but little greater than $\frac{1}{2}t$, instead of $\frac{2}{3}t$, the value obtained for wrought iron. The result is that the corresponding breaking stress, in the case of "All Moving Load," as deduced from these experiments, would be but little greater for steel than for wrought iron, although in the case "All Fixed Load" the breaking stress per unit of area for steel is one-third higher. This result would be somewhat unsatisfactory to bridge engineers, as, if acted upon, it would have the effect of requiring the area of a member subjected to a high ratio of moving load to be nearly as great with steel as with iron.*

It is to be noted, however, that much of the steel used in Wöhler's experiments was made about forty years ago.† The elastic limit of steel as now commonly used in bridge-building is probably higher, and there is no difficulty in obtaining a material having an elastic limit as high as two thirds the ultimate static breaking stress.

Hence it would appear that for the quality of mild steel, as now used for the construction of railway bridges, if the ultimate breaking stress per square inch be taken as 27 tons, the elastic limit may, for practical purposes, be taken as high as 18 tons. The rule for safe

* If the immediate effect of Moving Load be neglected, it would appear from the results of Wöhler's experiments that, under the condition of "All Moving Load," the breaking stress for steel might actually be less than that deduced for wrought iron, thus:

For wrought iron..... $u = \frac{2}{3}t$ and $t = 21$
For steel..... $u = \frac{1}{2}t$ and $t = 27$

The breaking stress for "All Moving Load" corresponding to these conditions would be, per square inch:

Wrought iron..... 14.00 tons
Steel..... 13.50 tons

† Wöhler's experiments were carried on for twelve years, from 1859 to 1870.

working stress may then be based on a value of u equal to $\frac{2}{3}t$* and the rule for steel may then take the same general form as that found suitable for wrought iron.

On these considerations the formula recommended for steel is:

Safe working stress in tons per square inch $= 9 - (5 \times R^2)$; corresponding with that proposed for wrought iron.†

The general results for steel, obtained by the use of this formula, are exhibited in Table No. 9.

TABLE No. 9.—Results for Steel. Obtained by use of Range Formula. Breaking Stress $= 27 - (15 \times R^2)$.

Nature of Compound Load. Ratio Percentage.		Breaking Stress for the Compound (Nominal) Load. Tons per Square Inch.			Coefficient to obtain Equivalent of Moving Load in Terms of Fixed Load.	Permissible Working Stress.	
			Apportionment of the Breaking Stress.			Factor of Safety = 3.	
Fixed Load.	Moving Load.	For the Total Load.	Due to Fixed Load.	Due to Moving Load.		Nominal Stress. Tons per Square Inch.	Effective Stress. Tons per Square Inch.
0	100	12.0000	0.0000	12.0000	2.2500	4.0000	9.00
2.5	97.5	12.7410	0.3185	12.4225	2.1478	4.2470	9.00
5	95	13.4625	0.6731	12.7894	2.0585	4.4875	9.00
10	90	14.8500	1.4850	13.3650	1.9091	4.9500	9.00
15	85	16.1625	2.4244	13.7381	1.7889	5.3875	9.00
20	80	17.4000	3.4800	13.9200	1.6897	5.8000	9.00
25	75	18.5625	4.6406	13.9219	1.6061	6.1875	9.00
30	70	19.6500	5.8950	13.7550	1.5344	6.5500	9.00
33.3	66.7	20.3333	6.7778	13.5555	1.4918	6.7778	9.00
35	65	20.6625	7.2319	13.4306	1.4719	6.8875	9.00
40	60	21.6000	8.6400	12.9600	1.4167	7.2000	9.00
45	55	22.4625	10.1081	12.3544	1.3673	7.4875	9.00
50	50	23.2500	11.6250	11.6250	1.3226	7.7500	9.00
55	45	23.9625	13.1794	10.7831	1.2817	7.9875	9.00
60	40	24.6000	14.7600	9.8400	1.2439	8.2000	9.00
65	35	25.1625	16.3556	8.8069	1.2086	8.3875	9.00
66.7	33.3	25.3333	16.8889	8.4444	1.1974	8.4444	9.00
70	30	25.6500	17.9550	7.6950	1.1754	8.5500	9.00
75	25	26.0625	19.5469	6.5156	1.1439	8.6875	9.00
80	20	26.4000	21.1200	5.2800	1.1136	8.8000	9.00
85	15	26.6625	22.6631	3.9994	1.0844	8.8875	9.00
90	10	26.8500	24.1650	2.6850	1.0559	8.9500	9.00
95	5	26.9625	25.6144	1.3481	1.0278	8.9875	9.00
100	0	27.0000	27.0000	0.0000	1.0000	9.0000	9.00

* For steel having a higher static breaking stress per square inch than 27 tons, the elastic limit would no doubt bear a lower ratio to the ultimate breaking stress than 2 : 3. But in such case the elastic limit would merely be lower relatively, not lower absolutely. A rule based on a breaking stress per square inch, of 18 tons for "All Moving Load," ranging to 27 tons for "All Fixed Load," would clearly not be less safe for a material, the breaking stress for which might range up to (say) 36 tons for "All Fixed Load." Some of the steel used in Wöhler's experiments showed a static breaking stress of more than 50 tons per square inch.

† In this formula the symbol R represents the proportionate range of stress, thus:

$$R = \frac{\text{Moving Load}}{\text{Fixed Load} + \text{Moving Load}} = \frac{\text{Range}}{\text{Total.}}$$

Rule Proposed for Adoption.

For the determination of the Safe Working Stress on any member of a bridge truss the following rule is proposed.

1. The Safe Working Stress for any member of a bridge truss is to be calculated by the following formulas:

Wrought Iron—

$$\text{Safe Working Stress} = 7 - (4 \times R^2). \quad \text{Tons per square inch.}$$

Steel—

$$\text{Safe Working Stress} = 9 - (5 \times R^2). \quad \text{Tons per square inch.}$$

2. In these formulas—

$$R = \frac{\text{Range of Stress}}{\text{Total Stress.}}$$

N. B.—The Range of Stress is to be taken as the difference between the lowest and the highest stress to which the bar is subjected.*

The Residual Fixed Load Stress is the balance of the Fixed Load Stress which remains unaffected by the application of the Moving Load, and which is therefore not subject to alternation.

The Total Stress is to be taken as the Residual Fixed Load Stress added to the Range of Stress.†

3. For a member subject to compression the area determined under Rule 1 is to be increased as may be necessary by the use of column formula.

4. For a member subject to a Range of Stress, partly in compression and partly in tension, the area required for the compression stress is to be ascertained under Rule 3, and the area required for compound stress is to be ascertained under Rule 1. The greater of the two areas thus found is to be adopted.

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* Hence, to obtain the Range of Stress, if these two extreme stresses be of the same sign (*i. e.*, both tension or both compression) the less is to be subtracted from the greater; if the two extreme stresses be of opposite signs (*i. e.*, one tension and the other compression) they are to be added.

† Hence, for cases where the range is between two stresses of the same sign (*i. e.* both compression or both tension) the Total Stress will be simply the initial Fixed Load Stress added to the greatest Moving Load Stress.

Where the range is between two stresses of opposite signs (*i. e.*, one compression and the other tension) the Residual Fixed Load becomes nothing, and the Total Stress is equal to the Range of Stress; the condition being that of All Moving Load.

CORRESPONDENCE.

HENRY S. PRICHARD, M. Am. Soc. C. E.—The writer has taken a special interest in this paper for the reason that in the specification he prepared for the use of the engineering department of the New Jersey Steel and Iron Company, as revised in 1895, a system of equating the comparative effect of moving and fixed loads is given, which is identical with the system termed "range coefficient" in the paper. The system as given in the specifications referred to is as follows: Mr. Prichard.

"To provide for impact and vibration an amount is to be added to the strains in each member in accordance with the following formulas:

"I = the amount to be added for impact and vibration.

"L = the combined strains from live load and centrifugal force.

"D = the dead load strain.

"For swing bridges and other movable structures, while in motion only, $I = 0.25\ D$.

"For counter strains $I = L$.

"For floor beam hangers and the riveted connections of the floor system $I = 1.25\ L$.

"For all other cases $I = L\,\frac{L}{L+D}$."

The writer derived this system by starting with Launhardt's formula and afterward modifying it to accord with the results of a study of all data, bearing on the subject, which were readily available. These data, however, did not include the valuable observations of the deflections of the bridges of India.

The independent derivation of this system by the author and the writer is a curious coincidence.

The subject of the paper involves two questions, one of fact and the other of method:

First.—What is the relative effect on a bridge member of the moving and fixed loads as compared with the sectional areas required?

Second.—What is the most practicable method of proportioning the member after the relative effect of the moving and fixed loads has been determined?

In discussing the first question, the author points out very clearly that a moving load causes a greater stress than a fixed load of equal magnitude, and that the repeated application and removal of the moving load makes the resulting stress produce a greater destructive effect than an equal amount of stress from a fixed load. He terms the stress caused by the moving load "the immediate effect," and obtains it by deductions from observations of deflections of bridges on the railways of India. He terms the result of repeated stresses the "cumulative effect," and obtains it by using a mean between Gerber's and Launhardt's deductions from Wöhler's experiments. Launhardt's formula

Mr. Prichard. in a modified form has been and is at present extensively used in America for proportioning bridges.

The modification consists in assuming that the dynamic effect, of which Launhardt's formula is supposed to take no account, is similar to that of repeated applications of the load without impact, and in further assuming that the greatest load which a bar can stand when applied an indefinite number of times, with an impact, such as is usual on railroad bridges, will be one-half the greatest fixed load. In Launhardt's formula, when applied to wrought iron, this allowed repeated load is two-thirds the allowed fixed load. The author, by collating the results of the observations of the deflections of the bridges on the railways of India, and deducing therefrom the average increase in the stresses from moving loads, has given something definite, in place of the arbitrary assumptions just referred to. This portion of the author's work has a great value, entirely separate from his subsequent combination of the results obtained with those of Wöhler's experiments.

This subsequent combination is made in a very logical way, and, if the results of Wöhler's experiments are strictly applicable to bridges, it is a rational one to make.

It was formerly, and is to some extent now, supposed that the seeming weakness of the metal from repeated stresses, as shown by Wöhler's experiments, was due to a general deterioration of the metal.

Professor Fidler contends that the seeming weakness can be accounted for by an increase in the stress from the dynamic action of the load.

Another theory is that there is no general deterioration, but that some of the numerous defects, or micro-flaws, gradually extend their weakening influence in an irregular plane of cross-section which ultimately becomes the plane of rupture, while the metal immediately adjacent to this plane remains, perhaps, wholly uninjured.*

It is probable that there is a weakening from repeated stresses in bridge members as well as increased stresses from the dynamic action of the moving load, but it is also probable that the degree of the weakening is less in them, as considerable intervals of rest occur between the passages of trains, than it was in Wöhler's experiments, where there were no such intervals of rest.

Experiments mentioned by Lord Kelvin† show that an elastic vibrator kept vibrating for several days through a certain range, comes to rest much quicker when left to itself than when set in vibration after having been at rest for several days, and then immediately left to itself.

Further, it is a matter of common experience that metal stretched beyond the elastic limit regains its elasticity after a rest.

* "The Materials of Construction," J. B. Johnson, M. Am. Soc. C. E., p. 537.

† "Elasticity," *Encyclopedia Britannica.*

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It seems probable, therefore, that, as regards the influence of repeated stresses, the effect of the moving load has been overestimated by the author. On the other hand, it is probable that he has underestimated the dynamic effect, for which provision should be made because he has dealt with averages.

The dynamic effect will vary greatly under different conditions, such as speed, condition of roadway, conditions of rolling stock, etc.

A structure should be designed for the greatest dynamic effect to which the class of structures to which it belongs may be liable, and not merely for the average, though, of course, the interval between maximums must also be considered in deciding upon unit stresses.

One example of the foregoing proposition should be especially considered. It is known that when there is, from any cause, a coincidence between the periods of the dynamic blows from the load and the vibration of a bridge, there will be accumulative vibration and increased stresses therefrom, in some cases the increase being considerable.*

It is probable that for a large proportion of railroad bridges accumulative vibration either does not occur or is quite small, and that for the remainder it only occurs occasionally. Such being the case, its average effect for a large number of bridges would be quite small as compared with its effect on a particular bridge.

In the present state of knowledge, it can hardly be said in advance that any particular bridge will be free from severe accumulative vibration; hence, to be on the side of safety, all bridges should be designed for the effect of severe accumulative vibration, even though comparatively few are affected, and the average effect small; but it may be admissible to use larger unit stresses than should be used if severe accumulative vibrations were certain to occur at frequent intervals.

The results which the author obtains, and which he terms the "results of experiments," are not quite in accord with his judgment when he recommends a system termed "range formula," differing somewhat therefrom, and points out its comparative advantages. The writer prefers the system of "range coefficients" to that of range formula.

Before discussing the relative merits of the different systems, it should be pointed out that for many bridge members not subject to alternate stresses, the minimum stress is not the stress from the fixed load alone, but this stress minus a counter-stress from the moving load. In applying the modified Launhardt formula, many designers have used the stress from fixed load in place of the minimum. On this account the Pennsylvania Railroad Company has added a note of warning in its specifications.

It does not appear from the paper what the author's views are re-

* "Vibration of Bridges," S. W. Robinson, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xvi, p. 42.

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garding such members. The writer advisedly uses the fixed load stress without distinction, both when it is and when it is not the minimum, in the formula given in his specifications; and in his subsequent discussion of the range coefficient system, to which his formula corresponds, this system must be understood as based on such use of the fixed load stress.

The deflections on whose averages the author bases his "immediate effect" curve (Fig. 1), would naturally, for the great majority of cases, be observed for the span fully loaded. When a span is fully loaded, the proportion of fixed load stress to moving load stress is nearly the same for all the members, and is nearly the same as the proportion of fixed to moving load. For those members, therefore, which attain their maximum stresses when the span is fully loaded, which for a span simply supported are the chords and end posts, the immediate effect obtained would seem to apply as an average. It can be noted from Figs. 1 and 2 that at about $\frac{\text{fixed}}{\text{moving}} = \frac{8}{92}$ the curves of immediate effect and total effect, termed by the author "standard curves," change their direction very suddenly, and that the only system that even approximately follows this change below this point is the range formula system, so that for those members whose maximum stress occurs when the span is fully loaded, the other systems ought not to be used if $\frac{\text{fixed}}{\text{moving}}$ is much less than $\frac{8}{92}$.

Using the Pennsylvania Railroad Company's specifications as a criterion, this would mean that the other systems should not be used for spans much less than 12 ft. As 12 ft. is a very short span, even for a stringer, the other systems would not often be ruled out on this account, and where they are specified the point can readily be covered by a qualifying clause.

The effect of accumulative vibration is occasionally so great for truss spans, even of considerable length, that it seems to the writer, that for moderate and moderately large values of $\frac{\text{fixed}}{\text{moving}}$, the standard curve, the range formula and the Bell-Robertson rule all give results which are too small. On the other hand, the fixed coefficient system and the modified Launhardt formula (referred to by the author as Launhardt's system) give results which seem unnecessarily high. The range coefficient system seems to be the best of any discussed. For very large values of $\frac{\text{fixed}}{\text{moving}}$ the results of the standard curve are probably too great, because for large values the dynamic effect, which is probably underestimated, is small as compared with the supposed weakening effect of repeated stresses, which is probably overestimated.

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An examination of Fig. 3, Table No. 6, shows that for large values of $\frac{\text{fixed}}{\text{moving}}$, say, for $\frac{67}{33}$ and greater, the unit stress allowed by the range coefficient system is slightly greater than by the standard curve, that allowed by the range formula system is slightly greater than by the range coefficient system, and those allowed by the other systems are less than by the standard curve.

The writer believes that the unit stresses prescribed by the other systems are lower than the facts seem to justify. He prefers the unit stress allowed by the range coefficient system, however, to the somewhat higher unit stress allowed by the range formula system.

The immediate effect of the moving load will vary with the suddenness with which it is applied, the impact with which it strikes the bridge, the accumulative vibration from the impact and the proportion of fixed to moving load; and these variations in the conditions will depend on the speed, character and condition of the rolling stock, and the length, character and condition of the bridge. In obtaining the immediate effect of the moving load, the author has noted the effect on each bridge considered as a whole only, and has considered only the variation in the proportion of fixed to moving load. Therefore, the result obtained for each particular ratio of fixed to moving load is the result for an average bridge, considered as a whole, for an average of all the other conditions. For bridge members whose maximum and minimum stresses occur when the bridge is only partially loaded, the average of the other conditions for any given proportion of fixed to moving would probably be quite different from the average for a bridge considered as a whole, because a small value of $\frac{\text{fixed}}{\text{moving}}$ can only occur for a bridge, thus considered, when the span is short while for members whose maximum and minimum stresses occur under partial loading a small value of $\frac{\text{fixed}}{\text{moving}}$ can occur when the span is long. For such members for a given $\frac{\text{fixed}}{\text{moving}}$ as the span increases, other conditions being constant, the load is applied more gradually. For very small values of $\frac{\text{fixed}}{\text{moving}}$, say $\frac{8}{92}$ and less, the sudden upward bend in the author's curve of immediate effect is probably due to the fact that for such values for bridges considered as a whole, the spans must be very short, and consequently not only is the moving load applied very suddenly, but the maximum impact comes from one blow of the driving wheels of the engine instead of from the aggregate of blows not simultaneously applied from several different units of the rolling load, as is the case for bridges of long span.

For the foregoing reasons the average immediate effect of the mov-

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ing load, for each particular value of $\frac{\text{fixed}}{\text{moving}}$, is less for members whose maximum and minimum stresses occur under partial loads than is indicated in the author's diagram, and if the value of $\frac{\text{fixed}}{\text{moving}}$ is very small it is much less. In view of these facts, the writer considers it wise, in applying the range coefficient system to such members, to use the fixed load stress instead of the absolute minimum, thus obtaining a smaller total effect, and to apply this system for even very small values of $\frac{\text{fixed}}{\text{moving}}$ without qualification.

For short spans, the character of the bridge plays an important part in determining the effect of the moving load. For pin bridges, owing to the play in the joints, the impact is greater than for plate and lattice girder bridges.

The writer has seen short pin spans and pin-connected floor beams in which the pin holes had become seriously elongated.

While it is hardly practicable to take the character of the bridges into account in devising a system for equating the effect of the moving load, it is advisable to limit the use of pins to the connections of fairly good-sized trusses.

There are many things, such as the condition of the bridge, the condition of the rolling stock, the amount of deflection, the amount of camber, etc., which help to determine the effect of the moving load. It is, of course, impossible to assign to each its value and to mathematically estimate their combined effect, but the fact of their existence and the uncertainty as to their effect makes it wise to keep well within what superficially seem to be safe limits, and to consider simplicity of greater value than hair-splitting refinements in selecting a system.

With the relative effects of the moving and static load known or assumed the question arises: What is the best method of proportioning? Too little regard is paid to this question. If the supposed facts as to the stresses were followed exactly, the nominal stress would vary with the dynamic action of the moving load, and the allowed unit stresses would vary with the weakening effect of repeated stresses. Precisely the same result can be accomplished, either by keeping the allowed unit stresses constant and providing for both the dynamic effect of the moving load and the weakening effect of repeated stresses by adding a sufficient amount to the nominal stresses, or the allowed unit stresses may be varied and nothing added to the nominal stresses.

The author's diagram, Fig. 1, shows the coefficients by which the moving load has to be multiplied for the various systems he discusses, if the first method is followed; while Fig. 2 shows the unit stresses to be used in the second method.

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Another excellent illustration and contrast of the two methods is afforded by a comparison of the specifications for bridges of the Pennsylvania Railroad Company with those of the Pennsylvania Lines West of Pittsburg. Both are based on the modified Launhardt formula before referred to, but the Pennsylvania Railroad Company uses the first method of proportioning, while the Pennsylvania Lines West of Pittsburg use the second.

The writer's fourteen years' experience in designing and detailing bridges has given him a decided preference for the first method; that is, the method which keeps constant unit stresses and provides for the total effect of the moving load by increasing the nominal stresses.

If the second method is followed, not only should the allowed unit stresses for the main sections be varied, but the allowed shear, bearing, etc., for the pins and rivets should likewise be correspondingly varied. This fact is so evident that it would seem useless to state it were it not for the fact that many, probably a large majority, of the specifications which vary the unit stresses allowed for determining the sectional areas of the main members, make no variation in the allowed shear and bearing for pins and rivets.

Probably few who have not designed by the second method, and consistently varied the unit stresses throughout, realize the labor involved in detailing by this method. In doing so, for every different member of the bridge, and in some cases, such as the stringers and floor beams for different parts of the same member, different bearing and shearing values for the rivets have to be used. This involves a calculation for each case to determine the allowed units, and, as it is not practicable to have tables for all possible unit bearings and shear, a further calculation to determine the value of the rivets in bearing and shear after the allowed units have been determined.

In using the first method, by which the entire effect of the moving load is provided for by increasing the nominal stresses, the allowed unit bearing and shear for the rivets and pins, as well as the allowed unit stresses for the main section, remain constant, and the work of detailing the connections is very much simplified as compared with what it is in using the second method.

In conclusion, the author is to be congratulated on having produced a paper which can hardly fail to considerably influence the future designing of bridges.

Mr. Thurston.

R. H. Thurston, M. Am. Soc. C. E.—The subject is one of supreme importance, not only to the builder of bridges, but also to every worker in metal, whether in construction of structures or machinery. Perhaps it is, if possible, more important to the latter than to the former; for the shake and jar and pound and inertia stresses, in heavy machinery, bring into the problem, in proportioning their parts, many

Mr. Thurston. forces difficult and often impossible to determine exactly, either in amount, direction or effect.

Investigations in two principal directions are gradually revealing the facts, data and laws upon which depend the resultant with which only the engineer is in the end concerned. The action of the dead load, as distinguished from the live load, is an inertia-effect. The action of repeated loads upon the material affected is a molecular phenomenon. The one is related to the weight and its motion simply, the other to the nature of the material attacked by that weight. The resultant effect, the strain produced and the risk involved by that strain must be anticipated by the engineer engaged in designing the piece, structure or the machine affected. In the railway bridge the load may vary from the sum simply of the dead and the live loads, as when the train is barely moving, to some approximation to the sum of the dead and twice the live load, as where the speed is such that the weight of the train may be taken as falling freely, and as ultimately conveying the stored energy of its fall through the extent of the sag of the bridge to the members thus overloaded, which parts, and the structure as a whole, presumably, must absorb all that kinetic energy before the fall can be arrested. It would seem that, as a matter of prudence, the engineer should assume in all such cases (in machine design certainly, and, perhaps, in the planning of parts of structures like bridges) that this maximum may be anticipated, approximately, on occasion, in the course of the life of the bridge or machine. If this be accepted as a principle of design in such cases, the computation of proportions of parts is to that degree simplified, and all errors will occur on the side of safety.

The effect of repeated stresses has now been so fully investigated that, although this field is undoubtedly still open for much profitable research, it may probably be assumed that enough of fact has been accumulated to give the designer of structures subject to repeated stresses, of the same kind at least, sufficient basis for his computations. The writer is inclined to sum the whole matter up by saying that the old idea, held by some of the most distinguished and ablest of investigators and designers, of making the elastic limit the unit upon which to base a factor of safety, has come to be established by the consensus of opinion of all modern investigators. The action of repeated stresses with reversal, involving, as it apparently does, even with small loads, constantly repeated and permanent disturbances of the molecular relations and arrangements, seems likely to demand prolonged experimental investigation and study of every class of material used in engineering. Yet it is now possible, with the more common materials, and for substantially important work, to establish limits, by reference to the experience of the century, which are those within which safety may be assured, practically, in all cases, even in the case

of machinery, with its constant reversal of stresses and strains. All these limit quantities, it may probably be said without hesitation, are found to be so related to the elastic limit of the material as to make that point the bench-mark of our work. The establishment of the elastic limit thus becomes the main requirement, it would seem, of the engineer's tests of materials, so far as used for purposes of construction of either structures or machinery. That being determined for the substance, and its relation to the breaking load under static stress, the judgment of the designer, guided by experience and research, enables him to proportion parts with entire safety and maximum economy in the use of material. Probably the majority of experienced designers of machinery would be satisfied with some such scale of factors of safety as given in Table No. 10:* Mr. Thurston.

TABLE No. 10.—Factors of Safety, Based on Elastic Limit.

Material.	Load.		Shock or repeated load in machinery.	
	Dead.	Live.		
Wrought iron—"mild" steel.....	1	2	3	Ratio of elastic resistance to working load.
Machinery steel,.....	1.5	3	5	
Tool steel.....	2	4	6	
Cast iron.....	3	6	8 to 10	

Where, as in bridge building, the movement of weights of great magnitude proportionally to dead load are to be often dealt with, the more precise methods illustrated by Wöhler, Bauschinger, Launhardt, Weyrauch, and by others since the pioneer work of these men was done, come into play. The paper presents an admirable summary of the investigations in this field to date, and the range formula therein given is, it seems, the best combination of simplicity with accuracy that has yet appeared. No one can feel hesitation in its use or distrust of the results when reduced to practice. The one direction in which further development is to be looked for is apparently the adaptation of the formula to use with all the now considerable variety of bridge material available and, in fact, to some extent used.

Formerly the one material obtainable, practically, for this purpose was wrought iron, which, when of really good quality, was a comparatively uniform and distinctly defined material in all its constructive properties. It had a tenacity of from 48 000 to 52 000 lbs. per square inch, and an elastic limit of not far from 18 000 to 20 000 lbs., and to insure uniformity and reliability of quality inspection could readily be made. To-day, iron is largely displaced by steel, and the quality of this latter constructive material may be made to range from that of a basic steel, reproducing substantially the properties

* "Materials of Engineering," Vol. ii, p. 341. R. H. T.

Mr. Thurston. of the softest grades of Swedish iron, through all the range familiar to the builder as the older standard bridge iron up to the various grades of harder steels, having tenacities of 70 000 to 80 000 lbs. per square inch and upward, and elastic limits rising from the lowest figures for Swedish irons and basic steels, perhaps 15 000 lbs., about a third the ultimate strength, up to 55 000 or 65 000 lbs., and two-thirds the ultimate tenacity, or more with hard steels.

Starting with the probably undeniable assumption that it is never desirable, in such work, to carry the loads above the elastic limits, under the most favorable circumstances, it is the necessary inference that the constants in the formula adopted must be made functions of the value of the elastic limit of the material selected for use. The formula must also adapt itself to the wide variety of constructive material now available, and actually in some degree made use of by the engineer. The general form of the expression being taken as here proposed,

$$S - a\,b\,R^2,$$

in which a is the stress to be allowed as a maximum, and b is the factor proportional to the assumed influence of the range-factor, R, the values of these quantities remain now to be adjusted to the quality of material to be used. The specified tenacity of the steel here considered as a typical quality seems low in consideration of the fact that the indications are, in some respects, very marked as promising the use of higher tenacities as improved manufacture gives better, more uniform and more resilient metals. Rail steel, for example, seems to be rising in standard, as also is steel for boiler plate, as the use of manganese is coming to be better understood.

The writer is very much inclined to expect that structural steel of all kinds will follow the course of rail steel, which has in the last few years changed from 0.25 or 0.35 carbon to 0.45 or 0.55; silicon from 0.05 up to 0.10; while the manganese has been, on the whole, rather reduced than otherwise by the improved materials and methods of steel making. In the common forms of steel, the relation of the elastic to the ultimate resistance is fairly constant at a ratio of six-tenths; but, should some of the later alloys come into use for structures, it may be expected that the proportion may raise to two-thirds. This will, presumably, make some difference in the margin of safety found desirable.

The particular fact to be noted in this connection is, that the elastic limit is the real gauge of the value of the material; whatever the nature or composition of the material, or whatever the character of the loads imposed. This being identified, the value of the member of whatever structure becomes settled. It is further to be noted, that the ratio of elastic limit to ultimate resistance and to safe load on the unit-area of section, for any given class of material, is substan-

tially constant for sound metal. It thus follows that, if it were possible to ascertain the elastic limit of any member or of any metal, the ultimate resistance would be computable, the safe load would become known, and the section suitable for a stated load, under specified conditions of loading, would become determinable. Mr. Thurston.

Ample experience and innumerable experiments have already shown that the loading of iron or steel to the elastic limit produces no permanent injury to the metal, and that the loading of any structure or of any member of a structure to the elastic limit, whether that limit is attained by static or dynamic loading, leaves the part practically as it was before loading. It simply takes out internal stresses up to that limit, and subsequent loads well within the limit do not affect either its method of yielding or its endurance.

All irons and steels employed in construction, when submitted to load of progressively increasing amount, up to and beyond the elastic limit, exhibit, on the stress-strain diagram representing the operation, a peculiar and unmistakable double curvature at the elastic limit, which exposes the location of that point with perfect accuracy, and the load sustainable there becomes constant, for a brief period, with continuing distortion. This marks, not only the elastic limit, but also the character of the material, and, if no other part of the diagram were obtained, the remainder of the curve could be easily and accurately laid down by anyone familiar with the material.

The testing of the piece up to the elastic limit thus serves quite as good a purpose as testing to destruction—with the decided advantage of saving the piece, leaving it as good as new and just as useful as a part of the proposed construction. This evidently applies to bridge rods or any parts in tension or compression, where the form and proportions of the piece are such as to permit the securing of a diagram such as is given by the ordinary test-piece. Placing it in test under loads approximating and passing the elastic limit, its ultimate and maximum carrying power becomes determined as completely as if it were actually broken. It can be then placed in the structure, and the engineer is as certain of its value and safety as if endowed with the gift of prophecy, as in fact he is in this instance. Every defect of composition, of make-up, or of molecular or mass structure, affects the elastic limit, and its rise above the normal intended indicates too great hardness and brittleness; its location at a lower figure than that proposed indicates either too soft a material or a faulty structure. If it has precisely the quality demanded by the specification, its elastic limit will be found at just the point anticipated, and the fact will be sufficient proof of compliance with the specification.

These principles were pointed out twenty years ago in a paper presented to this Society, March, 1878,* and illustrated by a set of

* *Transactions*, Am. Soc. C. E., Vol. vii, p. 53.

Mr. Thurston. figures which were practically representative of those obtained from the test of parts of the then famous Tariffville Bridge which had recently broken down with disastrous results.

In that instance, the test pieces from various members of the structure were found, after the accident, to have values indicated by the following figures, in which the loads are given in round numbers:

Resistances of Variously Strained Members.

	Elastic Limit.	Tenacity.
Sample No. 1.....	16 500	46 000
" 2.....	18 000	48 000
" 3	30 000	48 000
" 4.....	22 500	50 000
" 5.....	25 000	52 000
" 6.....	27 500	52 000
" 7.....	28 000	52 000
" 8.....	30 000	52 000
" 9.....	32 000	53 000
" 10.....	34 000	53 000

The extensions were usually small, ranging in the neighborhood of 15%, sometimes falling to 10 per cent.

The figures show that the original quality of the iron was good, soft and of low tenacity, but fine. Its elastic limit and ultimate resistance, as shown by Nos. 1 and 2, were about 17 000 and 45 000, or 48 000 lbs. per square inch. The extension was about 20 per cent. The abnormal relation between the elastic limits and the tenacity in Nos. 3 and 6, and higher numbers, shows overstrain, raising the original to a new and abnormal limit, and measures the heaviest load carried on each piece during its life history.

The record shows that the highest elastic limits thus revealed are found on parts taken from the structure at points nearest the section of the bridge at which the break occurred, this being found, as would have been anticipated as probable, though not certain, in advance of the investigation. The maximum elastic limit of the body of the rod corresponds accurately to the maximum strength of the threaded part of the piece, and measures the load on the structure, at that point, at the instant of breaking. This load was found to be about double that of the actual weight of the load producing the disaster, and this indicates either a shock, as by derailment, or a flaw in the rod at that point, either fact giving a sufficient explanation of the accident.

"It would be concluded that the ordinary loads, such as had been carried previously to the entrance upon the bridge of that which caused its destruction, never exceeded, in their straining action, 16 500 lbs. per square inch of section of tension rod. * * * This accident was therefore caused by the entrance upon the bridge of a load capable of straining the metal to about one-half of its ultimate strength, if

slowly applied, but which, in consequence of its sudden application, doubled that stress. Mr. Thurston.

"This sudden action may have been a consequence, either of its coming upon the structure at a very high speed, or the result of the loosening of a nut, or of the breaking of a part of either the bridge floor or of one of the trucks of the train. The latter occurrence, permitting the load to fall even a very small distance, would be sufficient."

Thus the key to the character and resistances of the material and of the members of the structure is to be found in the measurement of the elastic limit, and of its relation to the ultimate resistance of the metal. From this somewhat extended discussion, one is led to the conclusion that the values of the constants, in the proposed formulas for constructions in iron and steel, should be based upon the elastic limits rather than upon the ultimate resistances of the material.

If this be granted, the constant a, instead of being taken at a conventional fraction of the ultimate load under static stress, would be properly taken at a rational proportion of the elastic limit of the class of material to be used, as unity for soft irons and mildest steels, and perhaps three-fourths or eight-tenths for the harder grades. In that case, the formula would stand as at present for irons, while its constants would increase with rising values of the strengths of the metals, but at a slower rate. For hard steels, for example, where the elastic limit is sometimes found as high as two-thirds the ultimate, the higher of these proportions would be desirable, and the actual limit figure would not be far from 25 tons, over 50 000 lbs., per square inch. Nickel steels, should they ultimately become available for structural purposes, and manganese steels of the higher grades, would give figures still higher, if no obstacle be found to an increase of their hardening elements such as has been found practicable in some other directions.

One very important fact is often overlooked in the study of formula construction for structures: a permanent load may effect a progressive destruction of over-strained materials of some classes, and, as the writer showed by experimental researches of many years ago, "a so-called factor of safety of 2 may prove to be, after a time, a factor of unity." In other words, the indications of those experiments were that a factor of 2, based upon ultimate resistance, or 1 as based upon the elastic limit, might not certainly prevent fracture under static load, if it were sufficiently long imposed. Thus, taking 2 as the minimum limit of the factor for absolute dead loads, and 2 for the factor of ordinary risks, including what Holley called the factor of ignorance, it would seem that the product, 4, is a minimum for the most favorable cases, even where no live load is carried at all. This limiting value of the factor to be adopted must finally be increased in the proportion indicated by the researches giving as their outcome the formulas which have been discussed, and of which it would seem that the range-formula gives most satisfactory results as to form.

Mr. Thurston. In thus attempting to settle upon a factor of safety, upon either basis, with various qualities of material, such as are now available and actually more or less used in engineering, there always remains the uncertainty which is known to arise, even with the same material, as a result of known or unknown differences in the physical condition produced by differences in mechanical treatment. This is illustrated by the following experiment:

In the year 1883, the writer suspended a number of iron wires, under carefully adjusted and attached static loads, ranging, by differences of 5%, from 95% to 65% of their ascertained breaking loads under the ordinary tests, where the testing machine produces prompt rupture by a steadily augmented tension, unintermitted to the point of rupture. Two qualities of the same material were used; the one being set up hard-drawn as it came from the wire block, the other having been carefully annealed after the last reduction. The results of this experiment were as given in Table No. 11.

TABLE No. 11.—Endurance of Iron Under Dead Loads.

Per cent. maximum static load.	Time Under Load Before Fracture.	
	Hard, unannealed wire.	Soft, annealed wire.
95	8 days.	3 minutes.
90	35 days.	5 minutes.
85	Unnoted, but one or two years.	261 days.
80	91 days.	266 days.
75	Unbroken after several years.	17 days.
70	Same results.	455 days.
65	Same results.	455 days (probable jar).

Some of these wires were still unbroken in 1898, after fifteen years' loading; but, some slight oxidation setting in, the further prosecution of the experiment promises less certain deductions.

The significance of this research and its results becomes particularly notable when it is remembered that probably no two bars come from the rolls in the same state of hardening by mechanical strain or of annealing by the relative rates of heating and of cooling, in the process of production and subsequent treatment. The differences are probably never as great as those observed in the case above described, but they are of the same nature and may sometimes be considerable. In the case just related, the safety of the hard iron was evidently assured by a factor of safety of 2; while it is just as evident that a factor of 2 for the soft iron is, in fact, not certainly a factor of absolute safety at all. It still remains for special research to discover what bearing these facts have upon the safe proportions

of bridge-members and other constructions. At present it can only be said that great caution must be observed in the choice of material and in prescribing its treatment, as well as in the use of constants for the formula to be employed where computing for repeated and reversed loading. Mr. Thurston.

The writer would be inclined to adopt not over 5 tons, or, say, 12 000 lbs. per square inch for an average and good quality of what is recognized as the best bridge-irons, and would prefer them hard-rolled. For steels, perhaps one-half of the figure found for the elastic limit would serve for the value of the constant a until some of these still misty conditions and effects are more completely cleared up by further research. The writer would like to see the idea put in practice of testing construction material to the point of renewed elevation of the resistance under strain, beyond the elastic limit, to determine whether it shall be introduced into the structure. It would certainly, in the opinion of some of those who have carefully followed the investigation above related, prove a very satisfactory method of securing for the designer and constructor some confidence, if not certainty, that such members are actually capable of carrying fully the amount of load assigned them. Under the usual system, of testing by sample, the engineer always feels that it is extremely probable that he has tested sound specimens, while there have still more probably gone into the structure the exceptionally unsound and defective pieces received in the same lot. The discussion of these points by the writer, in earlier volumes of the *Transactions* and elsewhere, has furnished additions to the record which may, perhaps, prove of considerable interest in this connection.* Meantime the paper under discussion will certainly stand in those records, permanently, as a valuable and exceedingly complete and available presentation of the subject, up to its date.

J. A. L. WADDELL, M. Am. Soc. C. E.—The subject relates to one of the most important unsolved problems in the engineering profession, because, unless steel for all bridge members of all spans is strained about right, either metal is wasted or the ultimate danger limit is encroached upon in making bridge designs. The principal points upon which knowledge is necessary are the extent to which the metal in the various members of all approved types of modern bridges is strained by moving loads applied at different velocities, and the relation of the effects of loads so applied to the effects of the same loads applied statically. Mr. Waddell.

If such data were obtained, there could readily be constructed a diagram of percentages to add to live load stresses that are determined on the assumption of static application of loading, which would cover

* See Thurston's "Materials of Engineering," Vol. ii, Chap. x, and, particularly, Sec. 293 to 303, inclusive; also *Transactions*, Am. Soc. C. E., 1874, 1876, 1877, 1878, 1880, where are discussed the effects of time and load.

Mr. Waddell. the combined effects of impact, shock, jar, vibration, etc., all of which factors tend to increase the actual intensities of working stresses. The addition of these percentages would reduce the total stresses to equivalent static stresses, and would permit the adoption of one or two unit working stresses of each kind for designing. An allowance for unavoidable, small secondary stresses could either be included in this percentage diagram, or else this feature could be taken care of in determining the intensities of working stresses to use with equivalent static stresses.

The results of deflection observations on Indian bridges, quoted by the author, are certainly both interesting and valuable, but it is doubtful whether they will apply satisfactorily to modern American bridges, because the types of structures used in the two countries differ widely. Speaking generally, the weight of metal in an Indian bridge exceeds that in a corresponding American bridge, of the same span and loading, by from 60 to 100 per cent. This does not mean that the Indian bridge is so much stronger, stiffer, or better, but that the extra metal is simply wasted, its only good function being to absorb shock and impact. Such a result can be obtained just as well by using a material less expensive than steel, for instance, stone ballast in the floor. Again, Indian bridges are much more shallow than American bridges, and have far shorter panels and many more parts.

The writer has reason to think that the Indian experiments were made mainly upon spans as a whole, and not upon their component members; consequently, the results will apply only to chords as a whole and not to webs, for the writer does not agree with the author in the statement that "the results obtained from the deflection of the truss, as a whole, must to a great extent represent the average for all of its members."

Not only are hangers and other light web members much more subject to shock than are heavy chords, but also the light end panels of a bottom chord are probably somewhat more affected by impact, etc., than are the heavier chord members at and near mid-span. Experiment alone will determine the truth of this, and settle the vexed question of what are the various actual intensities of stress caused by live loads applied dynamically. The writer has recently expressed himself fully concerning this question of impact.*

The writer is of the opinion that a series of tests on actual intensities of working stresses for bridge members should be made. It would probably involve an outlay of from $100 000 to $200 000 to make these tests properly. An advisory committee of, say, five prominent bridge engineers should be retained to lay out, formulate and systematize the work, and to direct a working committee of three well-paid engineers who would devote their entire time and energies

* "De Pontibus."

to the investigation. Two members of the latter committee should be experienced experimenters and the other an expert bridge engineer. Mr. Waddell.

Most of the experimenting should be confined to modern structures of the most approved type; but, if time and funds would permit, some work should be done on bridges of the older types, many of which are still in common use. The beneficial results of such a series of tests are so apparent that it would be useless to discuss them.

The writer is surprised to see how large a portion of the paper is devoted to iron bridges, which are now things of the past, and how small a portion to steel bridges. The benefits to be derived from impact investigations are confined almost entirely to future bridges, which certainly will be of steel; because the practical railway engineer, in determining whether it is safe or not to continue to use an existing bridge, bases his judgment mainly upon the action of the structure under load, the excellence or weakness of its details, and its location in respect to traffic and shock, as well as upon the intensities of working stresses to which it is subjected.

It is questionable whether the author's method of tabulating or plotting his percentages of increase for live load stresses upon the basis of ratio of dead and live load stresses, is as correct or scientific as the more usual manner of basing them upon length of span covered by moving load; certainly it is by no means as convenient for the computor, who does not care to spend any unnecessary time in figuring ratios of loads, when he can find the percentage by simply glancing at a diagram, after noting the length of span to be covered for the greatest stress on the piece considered. In respect to the correctness of the author's method, let an extreme case be assumed for comparison, viz., a 200-ft. span of ordinary type, and a 100-ft. span so overloaded with ballast floor as to make the ratio of dead and live loads the same for the two spans. Would it be better to use the same percentage of increase for these two bridges or to make it greater for the short span than for the long one? Most engineers would certainly adopt the latter method.

The writer has never yet been convinced of the applicability of the results of the investigations conducted by German scientists on fatigue of metals to the proportioning of bridges. In order to obtain any results at all, the metal had to be strained far beyond the elastic limit, and the loads had to be applied every few seconds, while in bridges the metal never is (or, more strictly speaking, never should be) strained much higher than about one-half of the elastic limit, and the loads are applied at much longer intervals, thus giving the metal a chance to recover itself and return to its original state before the application of another load.

The determination of the value or values of intensities of working

Mr. Waddell. stresses is purely a matter of professional judgment, based upon experience in the field, and should not be relegated to scientists in their laboratories.

If the elastic limit is recognized as the true criterion of ultimate strength, and if the strain is no higher than one-half of this amount for equivalent static stresses due to ordinary loads, and 30% is added for the same, due to an unusual or practically impossible combination of all loads, including wind pressure, a sufficient margin will have been allowed to cover small secondary stresses, possible flaws in the metal and occasional dropping below average in elastic limit.

For medium steel having an ultimate tensile strength (measured on specimens) between 60 000 and 70 000 lbs. per square inch, and a least elastic limit of 35 000 lbs. per square inch, the writer in his latest specifications strains eye-bars 18 000 lbs. per square inch and built members in tension 16 000 lbs. per square inch.

The author evidently has more faith in metal than has the writer, for the former strains mild steel, having an average ultimate strength of 60 500 lbs. (27 English tons) per square inch, 12% higher than the writer does the medium steel. It is true that the author counts upon 40 300 lbs. (18 English tons) per square inch as the elastic limit of such mild steel; but the writer's experience in the use of metal does not indicate that it is safe to rely upon more than 33 000 lbs. per square inch as the elastic limit of such metal, to say nothing of the probable decrease in elastic limit between full-size members and test specimens. Standard specifications for soft steel allow the elastic limit to run generally as low as 30 000 lbs. per square inch, and sometimes even down to 27 000 lbs.

Referring to the Table No. 9, as a matter of curiosity, the writer has made a comparison of the nominal stresses involved, using the impact formula and intensities of *De Pontibus*, with those given by the author, using as a basis the bottom chords of single-track railroad bridges.

The results of the comparison are given in Table No. 12.

TABLE No. 12.

Span in feet.	Live load.	Impact.	Dead load.	Nominal intensity.	Stone's nominal intensity.	Percentage of difference.
100.....	4 366	3 844	1 630	12 144	14 280	+17.6
150.....	4 144	3 550	1 850	12 621	14 806	+17.3
200.....	4 030	3 303	2 130	13 102	15 365	+17.3
250.....	3 930	2 096	2 444	13 546	15 895	+17.3
300.....	3 860	1 930	2 820	13 966	16 484	+17.6
350.....	3 800	1 788	3 165	14 322	16 823	+17.5
400.....	3 760	1 671	3 500	14 631	17 150	+17.1
450.....	3 730	1 571	3 880	14 914	17 450	+17.0
500.....	3 700	1 490	4 226	15 169	17 713	+16.8
550.....	3 680	1 402	4 576	15 389	17 932	+16.5
600.....	3 660	1 331	4 930	15 585	18 120	+16.3
				Average difference.....		+17.1

It will be noticed that the author strains his mild steel nominally about 17% higher than the writer strains his medium steel, and that the variations in the differences are small. Mr. Waddell.

J. L. POWER O'HANLY, M. Am. Soc. C. E.—The deflection of a beam is the effect of an applied vertical force. Horizontal stress will not cause an increase of deflection. Hence, whenever a deflection is observed in a bridge, in excess of that due to the weight of the structure and its external quiescent load, the excess must be due to impact and acceleration, practically to impact. No other possible cause can be assigned for the phenomenon. For, were track and machinery perfect, the deflection of the bridge would be that due to the static load only, irrespective of speed of train. Hence, the observed difference is due to the imperfections of track and machinery. Increase of deflection may, therefore, be defined as the measure of the intensity of the impact. Mr. O'Hanly.

The paper is a useful addition to the existing scant literature on this very important subject, and will be hailed as of great assistance in solving the disputed question of the percentage of stress due to impact.

The writer believes that in designing railroad bridges of ordinary spans, the whole load, live and dead, should, in calculating stresses, be treated as all live load with a coefficient of 2.0. It is a safe rule, and takes some cognizance of the wants of the not distant future. In long-span bridges, however, with the dead load per linear unit clearly in excess of the train load, the line of demarcation between dead and live load may profitably be drawn, and a practice like that so well advocated by the author safely and economically introduced.

There can be little doubt that repeated applications of a load, particularly as it approaches the elastic limit of the material, impairs the strength, stiffness and efficiency of a bar and hastens its rupture. Universal experience confirms this doctrine, and brings the proposition within the definition of an axiom. To Wöhler is the great merit of bringing the problem within definitely assignable limits. The choice, however, of the expression fatigue, by scientists, to convey the idea, has been most unfortunate, almost bringing a sound and rational theory into ridicule, if not contempt. Fatigue will ever be associated with, and exclusively applied to, the effect of strain on animal tissue or living organism; nor is there any good reason to suppose that the feeling or sensation awakened thereby is aught akin to the severing or relative displacement of the molecules of iron or other inanimate substances.

In his examination of the tables, the writer is obliged to confess, even at the cost of unmasking his own ignorance, that he is unable, in many instances, to verify the author's figures.

Table No. 1.—It is to be regretted that the author did not find it convenient to accompany the paper with the data on which this table

Mr. O'Hanly. is founded, or at least as much thereof as would make it more comprehensible to those less fortunately circumstanced. The writer assumes that the results in the third column are based on the recorded deflection experiments, but how arrived at he is unable to surmise, except from the author's rather meager account.

Contrary to expectation, the writer finds that the results in column 4 are not strictly proportional to the corresponding figures in column 3, that is to say, the square inch ratio of

$$\frac{\text{Static breaking stress}}{\text{Compound load breaking stress}} < \frac{\text{Immediate percentage effect}}{\text{Live load percentage}}.$$

$$\text{For example, } \frac{21.00}{14.36} < \frac{147.4}{97.5}, \quad 1.46 < 1.51.$$

It is also less than the ratio of whole load. Thus:

$$\frac{21.00}{14.36} < \frac{147.4}{100}, \text{ or } 1.462 < 1.474.$$

The table gives the percentage of immediate effect of live compared with dead load, the breaking stress per square inch of section corresponding to such percentage of dead to live load, and the relative proportions of such stress per square inch due to dead and live loads respectively. Take, for example, the case of 50% dead and live loads. The breaking stress per square inch for this percentage is 20.60 tons, equally apportioned—10.30 tons—among the dead and live load components. As the writer understands the table, the immediate effect of 50 tons of live and dead loads respectively is equivalent to 101.94 tons of quiescent load, and it is surmised that the relative breaking stresses of the different kinds of load would be in like proportion, that is to say, 101.94 : 50 :: 20.6 : 10.1 breaking stress per square inch, due to dead load, and 10.5 tons to live, as against 10.3 in the table. If the exception be well taken, like reasoning applies to all the figures of columns 5 and 6.

Table No. 2.—The writer is unable to verify the figures in column 3 of this table, but fancies he has solved those of column 6, with the the aid of Gerber's formula—(Min. $S + \frac{1}{2}\triangle)^2 + 28\triangle = t^2$—as given by the author, from which the figures of columns 4 and 5 are readily deducible. Example—percentages of live and dead load respectively, 99 and 1. Then :

$$99 : 1 : : 14 : .14 = \text{Min. } S.$$
$$(0.14 + \tfrac{1}{2}\triangle)^2 + 28\triangle = 21^2.$$
$$0.02 + 0.14\triangle + \tfrac{1}{4}\triangle^2 + 28\triangle = 441.$$
$$\tfrac{1}{4}\triangle^2 + 28.14\triangle = 441 - 0.02 = 440.98.$$
$$\triangle^2 + 112.56\triangle = 1\,763.92.$$
$$\triangle^2 + 112.56\triangle + 56.28^2 = 1\,763.92 + 3\,167.44 = 4\,931.36.$$
$$\triangle + 56.28 = 70.22. \quad \triangle = 70.22 - 56.28 = 13.94.$$

TABLE No. 13.

Load Percentages.		Breaking stress due to live load.	Tabular numbers.
Dead.	Live.		
0	100	14.00	14.00
1	99	13.94	.....
2	98	13.89	.....
2.5	97.5	13.86	13.86
3	97	13.82	.....
4	96	13.76	.....
5	95	13.70	13.70
6	94	13.65	.....
7	93	13.58	.....
8	92	13.50	.....
9	91	13.43	.....
10	90	13.38	13.38
11	89	13.26	.....
12	88	13.22	.....
13	87	13.14	.....
14	86	13.08	.....
15	85	13.06	13.06
16	84	12.98	.....
17	83	12.90	.....
18	82	12.82	.....
19	81	12.65	.....
20	80	12.57	12.57
21	79	12.46	.....
22	78	12.39	.....
23	77	12.28	.....
24	76	12.19	.....
25	75	12.11	12.11
26	74	12.00	.....
27	73	11.89	.....
28	72	11.80	.....
29	71	11.69	.....
30	70	11.60	11.60
31	69	11.47	.....
32	68	11.35	.....
33	67	11.25	.....
33¼	66¾	11.23	11.24
34	66	11.15	.....
35	65	11.02	11.05
36	64	10.90	.....
37	63	10.79	.....
38	62	10.66	.....
39	61	10.55	.....
40	60	10.45	10.45

Columns 4 and 5 are easily deducible from column 6.

This table gives the percentages of cumulative effect of live compared with dead load, the breaking stress per square inch of section corresponding to such percentages of dead to live load, and the relative proportions of such stresses per square inch due to dead and live loads, respectively. Take, for example, the case of 50% dead and live loads. The breaking stress per square inch for this percentage is 18.20 tons, apportioned equally among the dead and live load components. The cumulative effect of 50 tons live and dead loads respectively is equivalent to 115.41 tons of quiescent load, the 50 tons of live load being equivalent to 65.41 tons ; and it is surmised that the relative breaking stresses per square inch of the different kinds of load

Mr. O'Hanly. are in like proportion. That is to say, 115.41 : 50 : : 18.20 : 7.88 tons breaking stress per square inch due to dead load, and 10.32 to live, as against 9.10 tons in the table. This reasoning, if correct, equally applies to the figures of columns 5 and 6.

Table No. 3.—The writer's mental darkness as to column 3 of this table, if not more intense, is just as persistent. With the aid, however, of Launhardt's formula, $a = 14\left(1 + \frac{\varphi}{2}\right)$, as given by the author, he has been enabled to verify the figures in column 4 from which those of columns 5 and 6 are readily deducible.

The table gives the percentages of cumulative effect of live load compared with dead load, the breaking stress per square inch of section corresponding to such percentages of dead to live load, and the relative proportions of such stresses per square inch due to live and dead loads, respectively. Take, for example, the case of 50% dead and live loads. The breaking stress per square inch for this percentage is 17.43 tons, equally apportioned in the table between the dead and live components. The cumulative effect of 50 tons live and dead loads, respectively, is equivalent to 120.45 tons of quiescent load; and it is surmised that the relative breaking stresses per square inch of the different kinds of load are in like proportion. That is to say, 120.45 : 50 : : 17.43 : 7.23 tons breaking stress per square inch due to dead, and 10.20 to live, as against 8.72 tons in table. The reasoning applies equally to all the figures of columns 5 and 6.

Table No. 5.—(*b*) Bell-Robertson Rule.

(1) As the writer understands this rule, which is given on page 491, it implies that in order to obtain the stress on the bridge equivalent to all static load, the whole load is to be multiplied by 1.5, and to the product the dead load added for the equivalent compound load, provided that the dead load is not less than half the live load. But should it be less than one-half, then half the live load shall be substituted therefor.

The tabular number corresponding to 2.5% dead and 97.5 live is 197.44. Applying the rule, the result becomes

$$100 \times 1.5 + \frac{97.5}{2} = 198.75,$$

as against 197.44 in Table No. 14.

(2) Construing the rule to imply that only the live load is to be multiplied by 1.5, and the dead load added to the product when not less than half the live, the result of the above would be

$$97.5 \times 1.5 + \frac{97.5}{2} = 195.0,$$

as against 197.44 in Table No. 14.

TABLE No. 14.

Mr. O'Hanly.

Percentage of Load.		Coefficient percentage for whole load.	Coefficient percentage for moving load only.	Tabular number.
Dead.	Live.			
2.5	97.5	198.75	195.00	197.44
5	95	197.50	190.00	194.74
10	90	195.00	180.00	188.89
15	85	192.50	170.00	182.35
20	80	190.00	160.00	175.00
25	75	187.50	150.00	170.70
30	70	185.00	140.00	166.67
33⅓	66⅔	183.33	133.33	150.00

It appears that a mean of the two comes nearer the tabular number than either.

(*c*) Range Coefficient Rule.—The writer's calculations agree with the figures in the table.

(*d*) Launhardt's Rule.—The author says:

"The use of this formula, with the values adopted, involves the following assumptions:

"(*a*) The coefficient to be applied to the Moving Load, to obtain its equivalent in terms of Fixed Load, is 2.0 for the condition of 'All Moving Load,' diminishing as the ratio of Moving Load to Fixed Load decreases, until it becomes 1.5 for the condition of 'All Fixed Load.' The variation in the value of the coefficient is directly in the ratio of Total Load to Fixed plus Total Load."

1. As the writer understands this rule, it implies that the coefficient 2 for all live load shall diminish in the ratio of total load to total load plus fixed load. That is to say, if the total load were 100 tons, of which the dead part 10, then 110 : 100 : : 200 : 181.81. If to this be added the dead load, the coefficient percentage becomes 191.81, with the tabular number 190.91. A table computed on this assumption gives the following results:

TABLE No. 15.

Load Percentage.		Coefficient percentage.	Coefficient percentage with dead load added.	Tabular number.
Dead.	Live.			
2.5	97.5	195.12	197.62	197.56
5	95	190.48	195.48	195.24
10	90	181.82	191.82	190.91
15	85	173.91	188.91	186.96
20	80	166.67	186.67	183.33
25	75	160.00	185.00	180.00
30	70	153.85	183.85	176.92
33⅓	66⅔	150.00	183.33	175.00

Mr. O'Hanly.

2. Assuming that the coefficient shall vary with each succeeding coefficient, the following result is obtained.

For example, suppose dead and live loads to be, respectively, 2.5 and 97.5 percentages. Then 102.5 : 100 : : 200 : 195.12. If to this be added the percentage of dead load, the sum gives 197.62. The corresponding tabular number is 197.56.

TABLE No. 16.

Load Percentage.		Coefficient percentage.	Coefficient percentage with dead load added.	Tabular number.
Dead.	Live.			
2.5	97.5	195.12	197.62	197.56
5	95	192.80	195.30	195.24
10	90	186.00	191.00	190.91
15	85	181.90	186.90	186.96
20	80	178.00	183.00	183.33
25	75	174.29	179.29	180.00
30	70	170.75	175.75	175.92
$33\frac{1}{3}$	$66\frac{2}{3}$	170.08	178.41	175.00
75	25	151.26	156.26	157.14
90	10	146.72	151.72	152.63

3. A closer approximation is obtained by assuming the percentages to vary by units. Thus, 101 : 100 : : 200 : 198.02. To which adding the dead load gives 199.02. Table No. 17 is constructed on this basis.

TABLE No. 17.

Load Percentages.		Coefficient percentage.	Coefficient percentage with dead load added.	Tabular number.
Dead.	Live.			
1	99	198.02	199.02	.....
2	98	197.05	198.05	.....
3	97	196.09	197.09	.....
4	96	195.14	196.14	.....
5	95	194.17	195.17	195.24
6	94	193.24	194.24	.....
7	93	192.32	193.32	.....
8	92	191.46	192.46	.....
9	91	190.55	191.55	.....
10	90	189.65	190.65	190.91
11	89	188.76	189.76	.....
12	88	187.88	188.88	.....
13	87	187.01	188.01	.....
14	86	186.15	187.15	.....
15	85	185.30	186.30	186.96
16	84	184.45	185.45	.....
17	83	183.61	184.61	.....
18	82	182.78	183.78	.....
19	81	181.97	182.97	.....
20	80	181.16	182.16	183.33

(*e*) Range Formula.—The writer has signally failed in all his attempts to verify the author's figures, as the following exhibit shows. Mr. O'Hanly.

All Moving Load.—$a = 21 - 12\ R^2 = 21 - 12 = 9$.

Then, $9 : 21 : : 100 : 233.33$

per cent. coefficient for all live load.

Example: Dead load 2.5%, live 97.5.

$$R = \frac{97.5}{100} = 0.975.\ R^2 = 0.95,\ \therefore$$

$$21 - 12\ R^2 = 21 - 11.4 = 9.6.$$

Then, $9.6 : 21 : : 100 : 218.75.$

Tabular number, 221.96.

TABLE No. 18.

Load Percentage.		Coefficient percentage.	Coefficient percentage with dead load added.	Tabular number.
Dead.	Live.			
2.5	97.5	218.75	221.25	221.96
5	95	206.49	211.49	212.09
10	90	186.17	196.17	195.74
15	85	170.32	185.32	182.73
20	80	157.66	177.66	172.07
25	75	147.37	172.37	163.16
30	70	138.95	168.95	155.56
33⅓	66⅔	134.04	167.37	133.33

Table No. 6.—The writer finds the figures in this table agree with his calculations, as far as verified.

Table No. 7.—The writer finds the figures of this table, as far as verified, agree with his calculations, taking the ratio of safe stress to experimental in Table No. 6.

Table No. 8.—This is merely a compilation in a single table of the results of Tables Nos. 5, 6 and 7 for the "Range Formula," the author's favorite rule.

Table No. 9.—This table is for steel structures what Table 8 is for wrought iron. The writer in this case encounters the same insuperable difficulty as for wrought iron, in finding the coefficient for steel to be multiplied into live load for reducing it to equivalent dead load.

The author well deserves the thanks of the profession for the thought and labor involved in the preparation of an interesting paper on a most important subject.

F. E. Turneaure, Assoc. Am. Soc. C. E.—In discussing the subject of live loads, the author makes certain statements which seem to the writer to be open to criticism. Mr. Turneaure

Mention is made of the live load of the text books as meaning a load suddenly applied in a vertical direction, and, therefore, not corresponding to the moving load of a bridge. Certainly no such notion

Mr. Turneaure.

of live load is conveyed by text books with which the writer is familiar, the term "live load" being used therein to indicate a moving load, and in no way connected with the suddenly applied load mentioned above.

The statement is also made that, neglecting the effects of vibration and deflection, the faster an engine runs in a horizontal direction the less its vertical effect. As the writer sees it, if the motion is strictly horizontal there is no vertical component, and, therefore, there can be no unbalanced vertical force acting on the body. This being the case, it follows that the upward pressure of the bridge on the moving body is at all times equal to the force of gravity acting downward, or, in other words, is constant for all velocities. If the deflection of the bridge causes the body to move in a path curved below the horizontal, there will be added to the effect of gravity that due to centrifugal force, which increases with increase in speed.

The author has done the profession a great service in separating and discussing so thoroughly the two effects of the moving load. These effects are too often treated apparently as one, but the distinction between them needs to be kept clearly in mind in discussing the subject of working formulas. It appears to the writer that the simplest way to treat the two effects is to add a percentage to the computed live-load stresses for impact, or immediate effect, and then to treat the stresses so found as the correct maximum live load stresses to be used in formulas taking account of fatigue, or cumulative effect. Practically the same results can, no doubt, be obtained in any given series of designs by the use of a single formula, as the author shows, but the use of a separate impact formula seems to be a more scientific procedure, and one that better enables the effect of each element to be appreciated and the judgment to be brought to bear on the selection of constants. Such a formula can also be more readily modified as our knowledge of impact increases.

The results of tests concerning the effects of live loads are very important, and it would be desirable to know more in detail of the tests whose results are summarized in the paper. A very important point is that of speed of trains used in the tests. It is to be presumed that the ratios plotted in Fig. 1 represent the total effect of the live load, including that of vibration, as compared with the effect of the same load moving at a very slow speed or standing still on the structure; but the writer would like to be assured on this point. It seems quite desirable to know, also, how far the maximum ratios exceed the averages plotted, as it is the maximum and not the average that must be provided for. The ratios given by the author are very much less than those observed by Professor Robinson*, and likewise much less than those found by the writer in a series of tests recently made on a

* "Vibration of Bridges," S. W. Robinson, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. xvi, 1887, p. 42.

number of bridges. The results of these tests agree in a general way with those made by Professor Robinson, and indicate that, for American practice at least, the percentages to be added for impact should be two or three times as great, for spans from 100 ft. to 200 ft. in length, as those given by the average curve in Fig. 1. Mr. Turneaure.

WILLIAM CAIN, M. Am. Soc. C. E.—The author has made a valuable contribution to the study of permissible unit stresses in the members of a bridge truss. From about 1 500 separate observations on deflections of bridges in India, the "immediate effect" of the moving load is obtained, it being assumed that the stress in any member is proportional to the deflection of the bridge—presumably at the center. Thus, for a fixed load, A tons per lineal foot and a moving load, B tons per lineal foot, Table No. 1, gives the factor by which B must be multiplied to give B', the corresponding static load that will cause the same deflection as B in motion. Mr. Cain.

There are doubtless additional stresses caused by impact and the resulting vibration (causing the rattling of the smaller members) that do not show in deflection, but these are neglected. For those engineers who do not believe that repetition of stress below the elastic limit causes any fatigue of metal, the cross-section of a member in square inches is logically found by dividing the stress due, not to the load $(A + B)$, but $(A + B')$, by 7, the safe unit stress allowed by the author, in tons per square inch.

The author agrees with many engineers in considering that millions of repetitions of stress should be allowed for by Gerber's or Launhardt's formula. To take the simpler of the two, or Launhardt's formula, the method now to follow to find the cross section of a lower chord member say, is first to find the stress in it corresponding to the load $(A + B')$ above and divide it by

$$\frac{14}{3}\left(1+\frac{1}{2}\,\frac{A}{A+B'}\right).$$

Table No. 3 gives the factor f by which B' must be multiplied, so that the stress corresponding to $(A + f\,B')$ when divided by 7 shall give the same cross-section as that just found. In fact, since the stress is proportional to the load per foot, the value of f can be found from

$$\frac{A+B'}{\frac{14}{3}\left(1+\frac{1}{2}\,\frac{A}{A+B'}\right)}=\frac{A+f\,B'}{7}.$$

Also, under "method of calculation" (page 481) is given the process of finding the coefficients in Table No. 4, so that B multiplied by the proper coefficient in this table shall equal $f\,B'$. This product added to A gives, thus, the load per foot to be treated as fixed load in finding the stress in the lower chord member supposed. On dividing

Mr. Cain. this stress by 7, the same area of cross-section is found as before. Thus, by use of Table No. 4, the stress resulting is simply divided by 7, which is simpler than the preceding method involving the direct use of Launhardt's formula.

The solution is correct for the chords, but it is to be inferred that the author supposes the same coefficients to apply to the web, which is erroneous. This would mean, in fact, that for a given fixed and moving load, the same unit stress is to be used throughout for tension in lower chord and every web member. The unit stress in a web member at the center, which varies from zero to its maximum, is thus to be taken the same as for the lower chord, which is inconsistent with Launhardt's formula.

To show where the error is introduced, it is to be first noted that the assumption on which Table No. 1 is founded is that the stress in any member of a bridge truss (leaving out the counters) is directly proportional to deflection. This may be granted.

The load, A fixed and B moving, is thus equivalent, by Table No. 1, to load $(A + B')$ fixed, and the stress can be found for this equivalent static load; but to get the cross-section of any web-member this stress must be divided by

$$\frac{14}{3}\left(1 + \frac{1}{2}\,\varphi\right),$$

where φ is not equal to $\frac{A}{A + B'}$ as for the chords, but to the actual minimum stress experienced in the passage of the moving load on the web-member divided by the maximum stress. But φ = Min. $S \div$ Max. S for a web member, varies from zero at the center of the truss to a value about equal that for the chords at the ends, and it is thus plainly inadmissible to take it the same throughout as for the chords.

It is equally inaccurate to take the true value of φ for a web member in applying Table No. 1, as the ratio of fixed to total load covering the entire bridge is used as the argument in the table, and this does not correspond to Min. $S \div$ Max. S for the web member. Besides Min. S in a web member corresponds to a different position of the moving load from that pertaining to Max S, whereas Table No. 1 refers to the same position. If a choice has to be made in order to utilize Table No. 4, then it is plainly best to use the latter method and replace the ratio of fixed to fixed and moving load in Table No. 4 by Min. $S \div$ Max. S for the web-member considered, for this gives the least cross-sections. This is the plan adopted in America, where the modified Launhardt formula is used for unit stresses.

It would seem, therefore, that the tables following Table No. 1 apply strictly only to the chords. For such members they are extremely interesting and instructive. Referring to Fig. 3, it will be observed that the straight line marked "Launhardt's system," is

much nearer the curve given by experiment than the one corresponding to Cooper's "fixed coefficient," and on account of its simplicity it is to be commended. Mr. Cain.

The author's formula, $a = 21 - 12\,R^2$, is supposed to allow, too, for increased stresses due to vibration, but there can be but little vibration in the chords, so that, although the author's formula is somewhat nearer the results of experiment than the modified Launhardt, still the simplicity of the latter appeals to the writer, and in the present state of knowledge respecting the effect of repetition of stress within the elastic limit, he sees no reason for adopting any more complicated formula, particularly noting what was said with respect to the additional approximations pertaining to web-members.

The formula for unit stress in pounds per square inch for wrought-iron ties, $a = 7\,500\ (1 + \varphi)$, was first proposed in *Van Nostrand's Magazine*,* the term $\frac{1}{2}\ \varphi$ of Launhardt's formula being empirically changed to φ to allow for all the influences of impact. The paper by J. M. Wilson, M. Am. Soc. C. E.,† shows that it was adopted with notable additions, before 1886, on the Pennsylvania Railroad, and it soon came into more general use. Later, the attempt was made by the writer, in view of Professor Robinson's experiments on deflection of American bridges,‡ to ascertain more nearly the scientific basis of the modification, and the results of the investigation were given in *Engineering News*,§ but the cream of the whole matter will be found in the discussion of the paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled "Some Disputed Points in Railway Bridge Designing."¶ Some of the leading points from that paper will now be given in a very brief form.

Launhardt's formula for breaking unit stress for millions of repetitions can be written,

$$\text{Unit stress} = \text{constant}\left(1 + \frac{1}{2}\,\varphi\right).$$

For a less number of repetitions, it is found, by working over the values given in the original experiments, that the coefficient $\frac{1}{2}$ is decreased and becomes quite small for a few hundred thousand repetitions, which is nearer the number to be provided for in most bridges than the former.

Taking A = stress in a lower chord member due to weight of bridge, B = stress due to live load at rest and Bp = superadded stress corresponding to the extra deflection due to the train's motion, then the actual stress in the member as the train rolls over is $A + B + Bp$.

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* November, 1877, p. 459.

† "On Specifications for the Strength of Iron Bridges," *Transactions*, Am. Soc. C. E., Vol. xv, p. 389.

‡ "Vibration of Bridges," by S. W. Robinson, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xvi, p. 42.

§ July 16th, 1887, *et seq.*

¶ *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 208.

Mr. Cain. Doubtless the coefficient φ in the Launhardt formula is less than $\frac{1}{2}$ for the average bridge, particularly where safe stress is considered below the elastic limit, but assume it at $\frac{1}{2}$ as a maximum, to account not only for repetition of stress, but also for the additional stresses caused by tremor not giving any vertical deflection, and write the safe stress in pounds per square inch,

$$b = 10\,000\left(1 + \frac{1}{2}\,\varphi\right),$$

whence the cross-section of the lower chord member will be,

$$\frac{A + B\,(1 + p)}{10\,000\left(1 + \frac{1}{2}\,\varphi\right)}.$$

Now if this same cross-section is found by dividing $A + B$ simply by the unit stress, $a = 7\,500\,(1 + \varphi)$, giving,

$$\frac{A + B}{7\,500\,(1 + \varphi)},$$

and this expression is placed equal to the other one, the following values of p are obtained:

Span in feet....	20	70	150	300	450
φ.....	0.10	0.20	0.33	0.50	0.66
p.....	0.30	0.28	0.25	0.22	0.20.

The values of φ given here are for old bridges, but as they sufficiently show the variations of p for modern bridges, it was not thought worth while to make the computations over again.

Professor Robinson's observations on deflection, previously referred to, show that the increased live load deflection due to the motion of freight and passenger trains at usual speeds varied all the way from almost nothing to 28% of the deflection caused by the same trains at rest.

In place of taking the average, it is certainly safer to take the mean of 25, giving the largest deflections, out of 193 transits observed, or 18.4%. This is considerably less than the maximum 28.

The bridges were open web bridges, 13 in all, of spans varying from 128 to 189 ft.

Thousands of observations on deflections of American bridges are needed to correspond in accuracy of results with the author's Indian experiments, but it is seen from the table above that the use of the formula, $(A + B) \div 7\,500\,(1 + \varphi)$, entails a value of p about 25% for the kinds of spans examined.

The experiments in India gave for $\varphi = \frac{1}{3}$ or $B = 2\,A$, about 0.07 (see Table No. 1), or very much less than the average of those in America. From present knowledge it would seem, then, unwise to adopt in America the coefficients given in this table.

If the formula 7 500 $(1 + \varphi)$ is changed to 8 000 $(1 + \frac{1}{8}\varphi)$ then p, for the bridges mentioned, becomes nearly 0.184, as found by experiment, but the unit stresses vary but little from the previous values, and awkward fractions had best be eliminated. Mr. Cain.

Fig. 3 shows that the extreme unit stresses given by the Launhardt system agree sufficiently well with experiment, and it seems needless to lower the coefficient of φ unless it is deemed best to allow for a less number of repetitions than, say, 40 million, when, as stated before, the coefficient may be lessened somewhat.

The previous results were deduced primarily for a chord member.

The formula for unit stress $= 7\,500\ (1 + \varphi)$ is likewise applied here, empirically to some extent, to the web; only $\varphi =$ Min. $S \div$ Max. S, is found for the proper positions of live load giving Min. S and Max. S.

Thus for counters and middle ties, $\varphi = 0$ and $p = 0.33$, or somewhat greater than the 0.25 corresponding to average spans. This is certainly not to be regretted, as it would seem that all counters, whatever the length of span, should have the smallest unit stress allowed, not only because they are suddenly strained from zero to their maxima, but also on account of their small size, permitting greater lateral vibration, which must be accompanied by increased stress.

For intermediate ties and posts, the unit stress increases pretty regularly from the center to the ends, as it undoubtedly should.

In the formula for unit stress, 7 500 $(1 + \varphi)$, the coefficient 7 500 for double rolled iron in links or rods, is to be changed to 7 000 for plates or shapes (rolled iron) and to 6 500 for rolled iron in compression. Corresponding formulas for steel are used, and for compression members the usual reduction in unit stress should be made by the accepted column formulas.

It may be objected that not only are the values of p, in the table on page 532, too high, but also that they do not decrease rapidly enough as φ increases.

The objection is not so serious as it would at first appear, as is brought out more plainly by the author's Fig. 3 than could be done by any tables.

Here, for the Indian experiments, the straight line for the modified Launhardt formula does not depart sufficiently from the curve given by experiment to invalidate the result as a practical solution, and the same may be said for the author's formula, $7 - 4\,R^2$, for safe unit stress in tons per square inch. The latter is a little nearer the truth, but it is a question whether the slightly greater accuracy cannot be sacrificed to gain the simpler modified Launhardt formula, particularly considering the very approximate nature of the solutions proposed for the various elements of the problem.

Mr. Moncrieff. J. M. MONCRIEFF, M. Am. Soc. C. E.—No more important subject for a paper, from the point of view of the bridge engineer, could be selected, but exception may be taken to the title, as the author deals exclusively with tensile stress, and the determination of safe compressive stress is entirely unnoticed, although compression members usually form a much larger percentage of the weight of a girder than the tension members. The highly important question of safe shearing stress on rivets subjected to variable loads is also untouched.

Whilst the weights of bridges of similar type certainly increase with the span, yet it is hardly correct to base thereon the assumption that the practice of bridge engineers in varying the unit stress according to the span is equivalent to an acknowledgment that the effect of moving load becomes relatively less as the fixed load becomes greater.

The importance of the span, rather than the weight of a bridge, in determining the probable influence of a live load receives recognition, according to the author's own references, in the practice of American engineers, under the special allotment system, in deciding on the allowable stress according to the span of the bridge, and the position of the member in the truss.

It is not at all definitely proved that "for any member of a bridge * * * the degree or percentage by which the effect of a ton of moving load is greater than a ton of fixed load varies for different ratios of moving load to fixed load," and proof is also required that "immediate" effect and "cumulative" effect can be in any sense looked upon as two distinct features of moving load influence on the materials of which a bridge is made, or, in other words, that dynamic influence and fatigue, in the usual acceptation of that term, have a definite separate existence. This is still an open question. With regard to the effect of horizontally moving loads on a bridge, in introducing the subject of Working Stress in Railway Bridges for discussion at one of the meetings of the Conference of the Institution of Civil Engineers, in London, in May, 1897, the writer pointed out that:

(1) "Although the influence of speed is difficult to estimate, yet *it does not follow that the maximum effect corresponds with maximum speed.* A familiar instance of this is found in the fact that a skater may frequently pass over ice that would not bear his weight for a single second if he were standing still, and, on the other hand, it is known that the maximum effect on an elastic structure does not occur when the load is standing still or going very slowly.

(2) "The deflection of a particular bridge under a train at 60 miles an hour may very possibly be no more, or even less, than when the train is going dead slow, and *there is probably, for any particular bridge, some particular speed at which a train will produce its maximum effect,* and this speed will evidently depend on the span of the structure, and it also appears quite possible that a train running at high speed may produce a relatively greater effect upon a bridge of considerable dimensions than upon one of short span."

Mr. Moncrieff.

This, of course, only refers to smooth running loads and is entirely exclusive of such influences as, (1) the action of the internal mechanism causing variations in the axle loads of locomotives; (2) the action of balance weights; (3) the dynamic effect of pitching and rolling of engines, arising partly from the two causes just mentioned.

These and other causes of additional stress require, and should be assigned, a special percentage of increase in effect as compared either with fixed or smooth running loads, or otherwise the same unit stresses for an electric or cable railway, as for the very different case of a railway with ordinary locomotives, with all the prejudicial influences of their internal mechanism, balance weights, pitching, rolling, etc., would have to be adopted.

The author's deduction that "the faster the engine (smooth running) moves in a horizontal direction, the less will be its effect in a vertical direction," is contrary to experience. If it were correct, horizontally running loads would be of less effect than fixed loads of equal amount as regards the production of deflection in a bridge or measurable strain in its members, whatever the relation between speed and span.

It is now also fully recognized by bridge engineers that it is absolutely necessary to keep in view the possibility and probability of trains running onto a bridge at great speed and being suddenly stopped on the bridge, and this is an occurrence which may happen any day on any bridge.

The author's difficulty in accepting the "live load" of the text books as assisting in the consideration of the influence of horizontally moving loads is somewhat difficult to understand, in view of the contingency just mentioned, and in view of the practically speaking rapid loading and unloading of the central diagonals of a lattice bridge under high-speed passing loads.

Referring to what may be termed the principal point of interest in the paper, *i. e.*, the deflection observations in India: In the first place, deflection observations on bridges with such proportions of depth to span most likely to be in use inIndia, but of which no particulars are given by the author, will no doubt give some idea of the effect of moving loads upon the flanges or booms, but such observations can hardly be taken as at all applicable to web members.

Attention also requires to be drawn to the fact that the "Standard Curve" given in the paper is an expression of averages only, and that for each point of the curve the maximum and minimum observations often differed widely from each other. It would be extremely interesting and valuable to have some idea of the actual variation.

Sir Benjamin Baker has very truly remarked that "it is the deviation from the average which really is so important in the design of engineering structures."

Mr. Moncrieff. It is not practicable to form any correct conclusions as to the influence of moving loads on bridges without full information as to span, type of bridge, proportions of bridge, character of details, speeds of trains, amounts of fixed and moving loads, and the full range of the results of the observations.

Again, referring to the division of moving load influence into "immediate effect" and "cumulative effect," as adopted by the author, it is by no means clear that the testing apparatus used by Wöhler, Spangenberg and Bauschinger (of similar type in each case) was such as to ensure that dynamic influence was completely eliminated, although shocks and jarring no doubt were avoided, but that is a very different matter. The writer is not aware that there are any records in connection with these tests of the "immediate" effect, *i. e.*, of the deflection or strain accompanying the stresses, and with the dimensions of the test pieces used it would be a very difficult matter to take observations of such small, but none the less all important, effects. An account of Wöhler's experiments, with illustrations of the testing machines, was published* in 1871, and a careful study of the machines and results of the tests affords good grounds for doubting the elimination of dynamic influence.

Under the head of "Results for Steel," the author states that for present-day mild steel of 27 tons tensile strength the elastic limit may be taken as high as 17 tons. This appears very doubtful. Does this figure refer to the primitive elastic limit of the material as it comes from the rolling mill?

One of the most important deductions to be drawn from Bauschinger's experiments is, that the primitive elastic limit of material, as it comes from the mill, is a most uncertain gauge of the powers of endurance of that material.

For instance, Bauschinger's endurance tests of wrought-iron plate, with a primitive elastic limit of 6.84 tons and ultimate strength of 25.2 tons, had powers of endurance approximately equal to that obtained with bar iron, having a primitive elastic limit of 11.8 tons to 14.8 tons and an ultimate strength of 26.7 tons per square inch.

Again, in one of Bauschinger's experiments on a Bessemer steel bar, he was able to cause the elastic limit to fluctuate all the way from 17.7 tons to 1.6 tons in tension, and from 9.65 tons to 3.24 tons in compression, by imposing loads alternately compressive and tensile, and the bar ultimately finished up with elastic limits equal for tension and compression, and amounting to 9.65 tons, which Bauschinger termed the natural elastic limit.

In comparing wrought iron and mild steel, it is absoluely necessary to have full details of their actual qualities, as the terms both cover material of a wide range of quality and physical character.

* (London) *Engineering*, Vol. xi.

The endurance tests of mild steel and wrought iron by Bauschinger and by Baker, on more modern material than that tested by Wöhler, agree in assigning a relative power of endurance to mild steel at least quite proportional to its greater strength, as compared with the wrought iron, and the latter was certainly of higher quality than would be generally used in ordinary plate or lattice girders, while the mild steel was of quite ordinary character. This is quite contrary to the deductions of the author. Mr. Moncrieff.

MANSFIELD MERRIMAN, M. Am. Soc. C. E.—The terms "immediate effect" and "cumulative effect," as used by the author, involve essentially the same ideas as those expressed by American engineers in the words "impact" and "fatigue." "Impact" includes the increase in stress due to the velocity of application of the live load and also due to oscillations. "Fatigue" means the lowering of the ultimate strength by a very large number of applications of the live load. The latter occurs only when the elastic limit is exceeded, provided there is no alternation from tension to compression, while the former occurs under every intensity of stress caused by the live load. The author's combination of impact and fatigue under the term total effect is the only systematic one that has come to the writer's notice, and the tables presented in the paper will undoubtedly be of great value in all future discussions. Mr. Merriman.

It is much to be desired that a detailed account of the deflection tests, from which the impact percentages were deduced, should be published. The increase in stress due to impact depends, not only upon the ratio of dead to live load, but also upon the length of the span and the velocity of the train; hence a discussion of these numerous experiments might add important information regarding the practical influence of these two elements.

The rules deduced by the author for the breaking stress, in tons per square inch are: $21 - 12\,R^2$ for wrought iron, and $27 - 15\,R^2$ for steel, in which R is the ratio of the live to the total load. The more usual method is to designate the ratio of the dead to the total load by ϕ. Thus $R + \phi = 1$, and hence $R = 1 - \phi$. Then the author's rules, expressed in terms of ϕ, are:

$$9\left(1 + \frac{8}{3}\phi - \frac{4}{3}\phi^2\right) \text{ for wrought iron,}$$

$$\text{and } 12\left(1 + \frac{10}{4}\phi - \frac{5}{4}\phi^2\right) \text{ for steel;}$$

and in these, $\phi = 1$ expresses the condition of all dead load, and $\phi = 0$ the condition of all live load, or in general $\phi = \dfrac{\text{Min. stress.}}{\text{Max. stress.}}$

The paper does not treat of alternating stresses where the minimum and maximum values are of different kinds, or where ϕ is negative. The rules are inapplicable to such cases, for, if the greatest tensile and

Mr. Merriman. compressive stresses are equal, then $\phi = -1$, and the breaking strengths given by the formulas are negative, or absurd. This appears to indicate that the rules are lacking in generality, for the perfect rules, when they are found, must be true for all values of ϕ from $+1$ to -1.

While the author's tables and rules are of great value for the case of the rupture of bridge members, under a very large number of repeated stresses which do not alternate from tension to compression, it may be doubted whether they retain this high value after division by the factor of safety of 3. Bridge members which are designed by the rules $7 - 4\,R^2$ and $9 - 5\,R^2$, for wrought iron and steel, respectively, are not stressed beyond the elastic limit, except in cases of accident; hence they are not subject to cumulative effect or fatigue, for all experiments appear to indicate that the properties of the metal are unimpaired by such stresses. In such cases, however, the stresses due to immediate effect, or impact, maintain their full influence. It appears, therefore, that the use of the author's formulas for the design of bridge members cannot be logically justified except as really introducing a factor of safety higher than the assumed 3. Until more perfect rules are established they should receive careful consideration, since they are a decided step in advance of many arbitrary methods now in use, and are apparently of greater generality than the formulas of Launhardt.

Mr. Wilson. JOSEPH M. WILSON, M. Am. Soc. C. E.—This paper is certainly a very interesting and practical one, directly in line with what is most needed in the determination and agreement upon a standard specification for bridges.

The application of systematic observations on deflections of bridges in India, under moving loads, to a comparison with methods of calculation already adopted or proposed, is most valuable.

While observed deflections cannot always be relied upon as an actual expression of the normal effect of moving load, the result being influenced by accidental or individual peculiarities of each particular structure, quality of workmanship, especially in riveted structures, etc., yet with such a number of observations at command, as in the present case, and the method of averaging adopted, they certainly form a fair, reasonable and most valuable addition to previous available data on this important question.

The systematic and exhaustive manner in which the author has carried out his investigations and comparisons gives great confidence in his results and leaves little to be desired.

It is quite evident that while one system of calculation, among those considered, may be deemed more comformable to the actual practical effects of the loading than another, and advisable to adopt for reasons of its own, yet, with the variations known to exist in the

best selected material, no one need feel uneasy with a bridge on account of its being calculated by either of these systems. Mr. Wilson.

The substitution of the term "moving" instead of "live" load, is a correction which should at once be adopted by every engineer, and the author is quite right in his explanation of the difference in meaning between these two expressions.

The writer looks with favor upon the "range" formula as offering advantages, in that it compels a lower working stress for short-span bridges where the fixed load is small as compared with the moving load, and permits a higher stress for longer spans where the reverse is the case. This is a matter in which, in the writer's opinion, the general practice will bear improvement, and some modification should also be made in the case of bridges near terminals, where the concentrated traffic of perhaps a number of lines accumulates, throwing on bridges an almost ceaseless service, the practical results of which, in America, at least, could not have been appreciated or anticipated a few years ago. America is advancing by vast strides, especially in its Eastern States, and that it is rapidly becoming an old country is shown in a marked way by the immense increase of local travel to and from its large cities.

J. B. JOHNSON, M. Am. Soc. C. E.—The author has conferred on the Society both a favor and a compliment by presenting this very thorough paper. Mr. Johnson.

As based upon the actual experiments of bridge deflections, his conclusions seem to be entirely justified. His use of a variable coefficient for reducing moving to an equivalent fixed load seems to be consistent with all known observations, and, as shown by Fig. 3, this use warrants a working stress for the ordinary ratios of fixed to moving loads some 25% greater than the assumption commonly made in the United States, which is that this coefficient should have the constant value of 2. His formula is simpler, also, and it may be written as follows:

$$p = \frac{f}{K}\left(1 - \frac{R^2}{2}\right) = \frac{f}{6}\left(2 - R^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where p = total working stress per square inch,

f = ultimate tensile strength of the metal,

R = ratio: $\dfrac{\text{live-load stress}}{\text{total stress}}$ in any member,

K = factor of safety.

When the American formula (with the constant factor 2 to reduce live to dead loads),

$$\text{which is } p = \frac{a}{1 - \dfrac{\text{Min. stress}}{2\ \text{Max. stress}}}$$

is put in the same form as equation (1) it becomes,

$$p = \frac{f}{K}\left(\frac{1}{1+R}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Mr. Johnson. As the author's formula (1) is at once more rational, or at least more in accordance with observed facts, and also gives about 25% greater working stresses, there would seem to be every reason why it should be used.

Mr. Stone. E. HERBERT STONE,* M. Am. Soc. C. E.—Before replying to the discussion the author wishes to thank the members for their very kindly and courteous reception of his paper. The subject is one which American bridge engineers may justly claim to have studied in all its bearings more thoroughly, more practically and more successfully than any other body of engineers in the world, and it is therefore specially gratifying to the author to find his contribution so favorably received.

In condensing the matter within reasonable limits for a paper, it was unfortunately necessary to leave out much in the way of explanation which would have anticipated some of the points raised in the discussion. The author's original work, however, is in the hands of many of the members, and a copy is in the library of the Society.

System of Adjustment.—It is now generally admitted that the effect of the Moving Load on a member of a bridge is more severe than that of the same weight as Fixed Load. It is further admitted that the degree or percentage by which the effect of Moving Load is thus greater than that of the same weight as Fixed Load is not in all cases the same, but becomes greater as the percentage of Moving Load in the Total Load increases. It is also agreed that for each particular case, if all the conditions could be ascertained and allowed for, there would be found a certain coefficient, by which, for practical purposes, the nominal Moving Load might be multiplied to obtain its equivalent or effective stress as Fixed Load.†

Opinions differ, however, as to the system under which this coefficient should be determined. Some engineers maintain that it should depend on the span of the bridge, while others are of opinion that it should depend on the proportion of Moving Load in the total compound load.

In this connection it may be pointed out that the system of making the coefficient vary with the span of the bridge would, if logically followed, assign the same unit stress to every member of the same bridge, irrespective either of its position in the structure, or of the ratio of Moving Load to Fixed Load. It is clear, however, that this system can only be considered approximately correct for the main

* The author has also forwarded supplementary discussions, designed to still further elucidate the paper, on "The Effect of Alternating Stresses," "The Factor of Safety," "The Trajectory Effect," "The Effect of Varying the Conditions," and "The Nature of Experiments Required"; all of which are filed for reference in the library of the Society.

† The author is aware that, on theoretical grounds, some engineers hold that the dynamic effect of the Moving Load cannot be represented properly by the static effect of a Fixed Load. For the practical purposes of bridge design the varying effects of the two kinds of load must be provided for on some definite system, and it is only a question of how this may most conveniently be done with sufficient accuracy for the purpose required. The practical side of the question is very clearly stated by Mr. J. A. L. Waddell in the first two paragraphs of his discussion.

booms—it is less suitable for the web bracing—and cannot apply at all to the cross-girders and rail-bearers.* Mr. Stone.

A rule based on this system must, moreover, often give inconsistent results, inasmuch as for bridges of the same span not only does the actual weight differ, but the relative weight of the Fixed Load as compared with that of the Moving Load depends greatly upon other conditions, such as gauge, the number of lines provided for, the amount of ballast, nature of flooring, and design of the structure generally.

It is also to be noted that a member of a small-span bridge may be under precisely the same conditions of loading as a member (either in the same or in some other position) on a large-span bridge, and in such case the size of the whole structure evidently does not at all affect the stresses to which that member is subjected.

It has further to be remembered that, of the total effect to be provided for, the cumulative effect is of very great importance; and it would be somewhat difficult to apply the results obtained by Wöhler and others for repetition and alternation of stress to a system under which the permissible unit stress is made to vary with the span.

There can be no doubt that to contend that the permissible unit stress should vary with the span is practically an admission of the principle that the permissible unit stress should vary with the ratio Fixed Load to Moving Load. For, if examined, this contention really means that in larger spans the mass of material in the structure itself is relatively greater as compared with the mass of material in the moving locomotive, and the effect of the locomotive in producing violent shocks and damaging vibrations is thereby to a greater extent modified, reduced or dispersed.

If an anvil be struck by a hammer, and the weight of the anvil as compared with that of the hammer be great, the amount of movement and vibration produced in the anvil will be relatively small. Further, if experiments were to be conducted with a series of hammers and anvils—so arranged that for each hammer and its anvil the relative weight differed, while in each case the total weight of the hammer plus its anvil remained the same—then it is evident that (other things being equal) the actual amount of movement and vibration would be greatest in that case for which the fraction $\frac{\text{Weight of anvil}}{\text{Weight of hammer}}$ was least.

On these considerations the author has preferred to make the variation in the coefficient depend on the proportion of Moving Load in the total compound load. He believes that this system is now approved by the majority of bridge engineers, and is adopted of necessity by all

* The cross-girders and rail-bearers in a long-span bridge would, of course, be treated as separate bridges of short span, with correspondingly lower unit stresses.

Mr. Stone. engineers who use Wöhler's results as embodied in the formulas of Launhardt or others. This system is, moreover, of universal application, whereas the system of variation by span applies to bridges only.

Fixed Load.—Where there are alternating stresses it is to be noted that a portion, or in some cases the whole, of the load due to the structure itself may become Moving Load, *i. e.*, a load which is applied and removed each time a train traverses the bridge. The Fixed Load to be dealt with in such cases is, therefore, not the actual total Fixed Load due to the structure, etc., but the residual Fixed Load, which remains unaffected by the application of the Moving Load, and which is therefore not subject to alternation.

Moving Load.—Mr. F. E. Turneaure has apparently misunderstood the author's allusion to the "Live Load of the text books." This special sort of "Live Load" is a load suddenly applied in a vertical direction, but without impact. Its immediate effect is exactly double that of an equal weight as "Dead Load." Regarding this the following extracts are quoted from well-known authors:

"Theoretical considerations, at first sight, might erroneously lead us to anticipate a greatly increased deflection under a rolling load, since they indicate the maximum deflection of an elastic beam under a suddenly imposed load to be double that of the final deflection after oscillations have ceased. The reason for this is obvious enough, for the resistance of the beam in deflecting to a certain extent, measured in foot-tons, must be equal to the work done by the load in its descent to the same extent; and as the latter is equal to the product of the load into the ultimate deflection, the former must be equal to the product of the mean resistance of the beam into the same deflection; hence the mean resistance of the beam must be equal to the load; and as the resistance is *nil* at the commencement of the bending, it must be equal to double the load at the termination" (B. Baker—"Long and Short Span Railway Bridges"—Edition 1873, page 92).

"It is well known that a load when suddenly imposed will produce a momentary deflection twice as great as that due to the same load at rest.

"This very important fact is illustrated every day in the action of an ordinary spring balance. Thus, if we take a common letter-balance, selecting one which works with very little friction, and if we suddenly place upon it (without shock or impact) a letter weighing 2 ozs., we know that the index-finger will at once be driven down the graduated scale until it points to something like 4 ozs., although after a few vibrations it will of course come to rest at the figure indicating the 'correct weight' of the letter. The extreme position of the index-finger upon the scale shows that the maximum stress, as well as maximum deflection of the spring, is twice as great as would be due to the same load at rest; and in the case of an elastic girder, a column, or a suspending rod, the same thing must take place irrespective of the amplitude of the vibrations, which in these cases may be so small as to escape observation." (T. Claxton Fidler.—"A Practical Treatise on Bridge Construction"—Edition, 1893, page 242).

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These quotations will, no doubt, make more clear the meaning of the author's remarks under "Live Load contrasted with Train Load."

The author sees no reason to alter his opinion that, "if we neglect the effects of vibration and deflection* and suppose the engine to run with absolute smoothness, it is clear that the faster the engine moves in a horizontal direction, the less will be its effect in a vertical direction." The illustration of the skater on thin ice will be familiar to every one. If the skater, having got up a high speed, puts his feet together and allows himself to glide on smoothly, he can traverse safely over ice which would give way at once under his weight if he stood still, and the faster his motion in a horizontal direction the less is the effect on the ice of his weight in a vertical direction. In other words, the effect of the weight of the skater on the ice is reduced instead of being increased by the speed at which he travels. If, however, instead of a smooth, gliding movement we have vertical oscillation, the conditions are much altered, and we find that ice which would just bear the weight of a man standing quietly upon it, would give way at once if he attempted to run on it, and the more violently he moved his feet and legs in running, the greater would be the effect in breaking the ice.

The point would be of no importance if it were not that some persons still believe that the "Live Load" of the text books corresponds in some way with the Moving Load of a locomotive, and it is quite common in older text books to find the statement that the effect of "Live Load" is just double that of the same weight as "Dead Load."

Two-Fold Nature of the Effect. – Mr. J. M. Moncrieff considers that proof is required that "immediate effect" and "cumulative effect" can be in any sense looked upon as two distinct features of Moving Load influence.

Mr. F. E. Turneaure, on the other hand, remarks as follows:

"The author has done the profession a great service in separating and discussing so thoroughly the two effects of the moving load. These effects are too often treated apparently as one, but the distinction between them needs to be kept clearly in mind in discussing the subject of working formulas."

Mr. Mansfield Merriman† says:

"I have read the paper with much interest and regard it as a valuable contribution to this important subject. Your distinction between 'immediate effect' and 'cumulative effect' is in particular calculated to simplify the theoretic discussion of dynamic action."

* The deflection of the girder introduces an entirely new element, namely, the "trajectory effect." It will be evident on consideration that this element in its action and mode of application is essentially different from the theoretical "live load" of the text books. To take the illustration of the skater, the nature of the trajectory effect would be well represented by the skater gliding at a high speed over the ice and suddenly falling on his knees. The trajectory effect is of the nature of a blow which may under certain conditions be very severe. In the experiments conducted at Portsmouth in 1848, under the direction of Commissioners Appointed to Enquire into the Application of Iron to Railway Structures, results for the trajectory effect upwards of three times as great as that due to the same load at rest were frequently obtained, and the highest recorded result was more than five times as great as that of the same load at rest.

† Letter to the author, dated January 25th, 1896.

Mr. Stone. *Immediate Effect.*—The method by which the "immediate effect" was deduced from the Indian experiments is described in the paper. The mass of detail from which these results were obtained was very great, and much of this material is no longer accessible to the author. The final plottings before the average curve was drawn are, however, still available, and can be sent to any one interested. The drawings have been shown to several engineers, who agree that the curve as reproduced in Fig. 1 is probably as good a representation of the results as could be obtained. The speed may be taken as about 40 miles an hour.

Mr. J. A. L. Waddell considers that the difference in design between Indian and American bridges might render it doubtful whether the results could be made applicable to American bridges. It would appear, however, that a rule under which the effective load on a member is made to depend on the proportion of fixed load and moving load would apply equally well to different designs, or at any rate sufficiently so for practical purposes.

Mr. H. S. Prichard notes that the curve for "immediate effect" changes its direction and assumes a different character for the higher ratios of Moving Load. This feature is due to "trajectory effect." What is believed to be the normal position of the curve is shown by dotted lines in Fig. 1.

The author ventures to think that Mr. William Cain has misunderstood the mode in which the results were worked out. The Indian experiments were by deflection only, and the girder was in each case treated as a whole. The results obtained from any particular girder were not considered applicable to every member of that girder, but to any member (either in that or in any other girder) in which the ratio $\frac{\text{Fixed Load}}{\text{Moving Load}}$ was the same. For example, suppose in a particular class of girders the ratio $\frac{\text{Fixed Load}}{\text{Moving Load}} = \frac{20}{80}$, and that for these girders the experiment showed that the immediate effect of the Moving Load was 113% of that due to the same weight as Fixed Load. This 113% "equivalent" would be considered as suitable to any member for which the same ratio $\frac{\text{Fixed}}{\text{Moving}} = \frac{20}{80}$ applied, but not necessarily to any individual member of the particular girder tested.

The author, however, does not attach any particular importance to the Indian experiments, and has only made use of them in his paper because there were no others of such an extensive nature available. He wishes them to be considered merely as giving general average results and will gladly see them superseded later on by a better series of experiments made on scientific principles by skilled observers.

Results for Steel.—It has been remarked in the discussion that the greater part of the author's paper is devoted to structures of wrought

iron, and that only a comparatively small portion is occupied with the results for steel. The reason for this will be obvious on consideration, as the greater portion of the argument applies both to iron and to steel and could be used for either material indifferently by a mere alteration of the figures representing the stresses and coefficients. It was therefore considered better to let the argument run through continuously for one material, *viz.*, wrought iron, and then show how the results thus obtained should be applied to steel. The only alternative to this method of dealing with the subject would have been to have given the greater portion of the paper twice over. Mr. Stone.

Some engineers consider that the author in taking the elastic limit of steel as used for modern bridge work at two-thirds the ultimate breaking stress has taken a higher ratio than would ordinarily be obtained in practice. In this connection the following is quoted from *Engineering News*, February 28th, 1895, page 135:

"It has frequently been claimed by structural steel makers that in drawing specifications the elastic limit should not be specified to exceed half the ultimate strength, and that at least 10 000 lbs. variation should be allowed between maximum and minimum limits of ultimate strength. Some recent tests, however, have shown that there is no difficulty in procuring steel with an elastic limit equal to two-thirds the ultimate strength, and that a difference of 7 000 lbs. between the specification limits for ultimate strength can be met without difficulty. We are indebted to Mr. George H. Thomson, M. Am. Soc. C. E., Consulting Engineer, for the accompanying condensed record of seventy-one tests upon 600 tons of bridge steel made to his specifications and rolled in January, 1895, by the Pennsylvania Steel Company and Carbon Steel Company and inspected by Mr. A. C. Cunningham, of the American Engineering and Inspection Association, of Albany, N. Y. The steel consisted of plates and angles $\frac{3}{8}$ in. to $\frac{3}{4}$ in. thick, for use in six spans on the Central Vermont Railroad, and no difficulty was found in meeting the high requirements of the specifications, only a very few pieces tested falling below the requirements. The lowest elastic limit found was 61.1% of the ultimate strength. The condensed record of the tests is as follows:"

ABSTRACT OF RESULTS.*

	Static breaking stress. Tons.	Elastic limit. Tons.	Elastic limit ratio. Tons.
Pennsylvania Steel Co.			
With lowest elastic limit	26.295	16.571	63.02
With greatest ultimate strength	28.362	18.147	63.98
With least ultimate strength...	25.491	17.424	68.35
Carbon Steel Co.			
With maximum elastic limit...	25.518	19.411	76.07
General Results.			
Highest results	28.362	19.411	68.44
Lowest results.....	25.491	16.571	65.01
Average	27.057	18.108	66.93

* In the original the stresses are given in pounds, and have here been converted to their equivalents in tons for convenience of comparison with other values given in the paper. The table given here is a condensed abstract of that given in *Engineering News*.

Mr. Stone. Hence it would appear that for the quality of mild steel, as now used for the construction of railway bridges, if the ultimate breaking stress per square inch be taken as 27 tons, the elastic limit may for practical purposes be taken as high as 18 tons; and, in dealing with cumulative effect the rule for safe working stress may then be based on a value of u equal to $\frac{2}{3}$ t.

Members in Compression.—If the effect of bending be sufficiently provided for by the design and by the use of a suitable column formula, there appears to be no reason for adopting a lower permissible working stress for members in compression than for those in tension. The fact that the ultimate breaking stress in tension may be somewhat higher than the ultimate crushing stress in compression is here not of much importance compared with the fact that (if the effect of bending be provided for), we may, for evident physical reasons, work to a higher unit stress in compression than would be safe or prudent in tension. For a member in compression the "cumulative effect" is a matter of comparatively small importance, and many of the reasons for adopting a factor of safety as high as 3.0 for members in tension do not apply to members in compression. There can be no doubt that, if the bending effect be adequately and separately provided for, the material in compression may safely be subjected to a greater working load than would be admissible in tension.

Application of Results.—When the permissible stress under different conditions has been determined, the question at once arises as to the method by which the results thus agreed upon shall be applied in practice.

Take for example the case of a member of a truss of wrought iron conditioned as follows:

	Tons.
Fixed Load	20
Moving Load	80
Total	100

and let it be assumed that the following have been ascertained or decided upon as suited to these conditions—

	Tons per square inch.
Safe Working Stress:	
For all Fixed Load	7.00
For the compound (nominal) load	4.44

and that the coefficient is 1.72.

It is evident that we may proceed to ascertain the area required for the member either by the effective stress obtained by the use of the coefficient thus:

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Effective Stress Method.

Effective stress $20 + (80 \times 1.72) \ldots = 157.6$ tons.
Safe working stress per square inch $= 7$ tons.
Area required $\frac{157.6}{7} \ldots = 22\frac{1}{2}$ sq. ins.

or from the nominal stress thus:

Nominal Stress Method.

Nominal stress $20 + 80 \ldots = 100$ tons.
Safe working stress per square inch $= 4.44$ tons.
Area required $\frac{100}{4.44} \ldots = 22\frac{1}{2}$ sq. ins.

Under the Effective Stress method the same safe working stress (*viz.*, 7.0 tons per square inch) would be used for every member of the bridge and applied to the effective stress. The coefficient would be obtained from a table, or from a diagram similar to Fig. 2.

Under the Nominal Stress method the permissible nominal stress per square inch would vary, and would be obtained from a table, or from a diagram similar to Fig. 3.

In practice it is probable that in either case the rule or formula would in the first instance be used to construct a table or diagram, and that such table or diagram would for general work be used in preference to the formula.

The Effective Stress method appears to be not only the more scientific, but is also more convenient, as has been very clearly shown by Mr. H. S. Prichard.

Mode of Calculation.—Mr. J. L. Power O'Hanly has raised some questions regarding the mode of calculation adopted.

Table No. 1.—Taking for example the case cited by Mr. O'Hanly in which the percentage ratio is $\frac{\text{Fixed}}{\text{Moving}} = \frac{2.5}{97.5}$ and suppose for the sake of illustration that these figures represent tons, we have then in this case a loading as follows:

	Tons.
Fixed Load	2.5
Moving Load	97.5
Total load	100.0

It is found by experiment that for this ratio of loading the immediate effect of the Moving Load is 1.474 times as great as that due to the same weight as Fixed Load.

Hence the equivalent of the Moving Load in terms of Fixed Load is

$$97.5 \times 1.474 = 143.715 \text{ tons;}$$

and the total effective load is

Actual Fixed Load.		Equivalent Moving Load.		Total Effective Load.
2.5	+	143.715	=	146.215 tons.

Mr. Stone. That is to say, the 100 tons of load, made up of 2.5 tons fixed and 97.5 tons moving, is equivalent, as far as stresses on the structure are concerned, to 146.215 tons all Fixed Load. Taking 21 tons per square inch as the breaking stress for all Fixed Load, the breaking stress per square inch for this Nominal Load of 100 tons will be $\frac{21 \times 100}{146.215}$ = 14.3624 tons per square inch.

Taking a factor of safety of 3.0 we get permissible working stresses as follows:

	Tons per square inch.
For the Effective Load	7.0000
For the Nominal Load	4.7875

and either $\frac{146.215}{7}$ or $\frac{100}{4.7875}$ will give the area required, *viz.*, 20.89 sq. ins.*

As regards the apportionment of the breaking stress (taking the same case) we have ascertained that a bar 1 sq. in. in section will, under the conditions assumed, break with a Nominal Load of 14.3624 tons. This load is made up of 2.5% Fixed Load and 97.5% Moving Load. Hence the Nominal Load which will break the bar of 1 sq. in. in section is made up as follows:

	Tons.
Fixed Load	0.3591
Moving Load	14 0033
Total	14.3624

But to obtain the actual effective load we must multiply the nominal moving load by the coefficient 1.474, and we then get the following as the actual effective load which will break the bar of 1 sq. in. in section:

	Tons.
Actual Fixed Load	0.3591
Effective Moving Load	20.6409
Total	21.0000

Tables Nos. 2 and 3.—A similar explanation will apply to Tables Nos. 2 and 3 for the cumulative effect of the Moving Load as deduced by the use of Gerber's parabola and Launhardt's formula, respectively. The results have all been worked out to four places of decimals, and the nearest figures to two places of decimals have been entered. The figures have been re-checked by the author and found correct.

Table No. 5. Bell-Robertson Rule.—In regard to the results given in Table No. 5 for the coefficients for the Bell-Robertson rule the process of calculation is as follows:

* It is to be remembered that these figures are for "immediate effect" only. Cumulative effect is not here taken into account.

Taking the example selected by Mr. O'Hanly, *viz.*—$\frac{\text{Fixed}}{\text{Moving}} = \frac{2.5}{97.5}$; Mr. Stone.
suppose the total load to be 100 tons.

Then under the rule we have:

	Tons.
Moving Load (97.5) multiplied by 1.5	= 146.25
Add a minimum Fixed Load equal to half the Moving Load $\frac{97.5}{2}$	= 48.75
Total Effective Working Load	= 195.00
Deduct actual Fixed Load	2.50
Effective Moving Load	= 192.50

The coefficient for obtaining the equivalent Effective Load due to the nominal Moving Load will therefore be $\frac{192.50}{97.5} = 1.9744$.

Table No. 5. Launhardt's Formula—The remarks about the law of the variation of the coefficient for the modified Launhardt formula will, perhaps, be made more clear if expressed as a formula thus:

$$\text{Coefficient} = 1 + \frac{\text{Total}}{\text{Fixed} + \text{Total}}.$$

This is, however, merely a matter of general interest, as the coefficients given in the table were calculated independently, not by this rule.

Range Formula.—Taking the example selected by Mr. O'Hanly, the mode of calculation is as follows:

	Tons.
Fixed Load	2.5
Moving Load	97.5
Total	100.0

Here $R = 0.975$ and $R^2 = 0.9506$.

By range formula: Breaking stress $= 21 - (12 \times R^2)$
$= 21 - (12 \times 0.9506)$
$= 21 - 11.4072$
$= 9.5928$

That is to say, that under the conditions assumed, a bar 1 in. square will break under a nominal load of 9.5928 tons made up as follows:

	Composition of Compound Load. Percentage.	Tons.
Fixed Load	2.5	0.2398
Moving Load	97.5	9.3530
Total	100.0	9.5928

Mr. Stone. But the breaking stress for this same bar 1 in. square, under all Fixed Load, would be 21 tons.

Hence the nominal Moving Load of 9.3530 tons under these conditions produces an effect on the bar as severe as that due to 20.7602 tons of Fixed Load, thus:

	Tons.
Actual Fixed Load.....	0.2398
Effective Moving Load.....	20.7602
Total.....	21.0000

Under these conditions, therefore, the coefficient to be applied to the Moving Load to obtain its equivalent in terms of Fixed Load will be—

$$\frac{20.7602}{9.3530} = 2.2196$$

Selection of a System.—For the reasons given in the paper, the author believes that the range formula gives a closer approximation to the actual facts than any other of the six systems which have been discussed.* In form and principle it may be compared with the modified Launhardt formula, but it will probably be agreed, on an inspection of Fig. 3, that the curve given by the range formula is more satisfactory than the straight line of the modified Launhardt formula.

If, however, something simpler be preferred, the author would invite attention to the range coefficient system. It is exceedingly simple to use in practice, easily remembered, and, next to the range formula, gives better results than any other system. It will be remembered that under this system the coefficient is obtained by merely adding 1.0 to the decimal giving the proportion of Moving Load to Total Load.

Examples :

Moving Load 0.9 of Total Load.
Then coefficient = 1.9.
Moving Load 0.66 of Total Load.
Then coefficient = 1.66.

The author notes that Mr. H. S. Prichard considers this (the range coefficient) the best system that has been discussed.

Application to American Practice.—Mr. J. A. L. Waddell finds the results for permissible nominal stress as given in the paper higher than he would consider advisable to adopt. On the other hand, Mr. O. Chanute, on behalf of himself and Mr. C. L. Strobel, remarks:† "We are both much pleased with your work, and find that it differs but little from current American practice."

* It is probably in fact a closer approximation than even the results given by experiment, because it makes a certain allowance for the effect of vibration and concussion.

† Letter to the author, dated June 20th, 1896.

Mr. Stone.

After examining a number of modern American specifications the author would suggest for consideration the following modifications of his range formula as likely to be found suitable in general American practice.

Wrought Iron.

Pounds per square inch.

$$\text{Safe Working Stress} = 16\,000 - (9\,000 \times R^2)$$

Medium Steel.

$$\text{Safe Working Stress} = 20\,000 - (11\,000 \times R^2)$$

Should the results obtained by these formulas, when plotted, be found not to be quite satisfactory a revised line can then be sketched in to pass through the points required and an amended formula made accordingly.*

Modifications in the Formula.—The author wishes it to be understood that he does not attach any special importance to the precise figures given with his proposed range formula. This formula has been merely suggested by him as a means of obtaining a curve (see Figs. 2 and 3), which will represent the known facts better than those obtained by other rules which have been proposed.†

In any case, however, it will no doubt be agreed that the right principle is first to draw a curve in accord with the facts of the case as far as can be ascertained, and then to devise a formula which will represent this result as simply as may be and with a sufficient degree of accuracy for practical purposes.

Experiments Required.—With the engine standing at rest there is no difficulty in ascertaining the stress which should come on each member of the bridge owing to the weight at the wheel-tread of each wheel; and although we are aware that (owing to peculiarities of construction or to improper treatment during erection), each member does not, as a matter of fact, receive just the exact stress due to it by calculation, yet we are satisfied that for all practical purposes the results given by calculation are a sufficiently near approximation. But when we have to determine the special extra effect on each member of the bridge due to the condition of the load being changed from "engine-at-rest" to "engine-at-full-speed," the problem at once becomes exceedingly complex. Among the elements which affect the result, we have, for example:

(*a*) Temporary excess loads for particular wheels due to the pitching of the engine.

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* It will be observed that these formulas are based on the following assumptions:

	Wrought Iron per square inch.	Medium Steel per square inch.
Breaking Stress.....	48 000 lbs. = 21.43 tons.	60 000 lbs. = 26.79 tons.
Safe Working Stress	16 000 " = 7.14 "	20 000 " = 8.93 "

Tons per square inch.

† For wrought iron, the formula, Safe Working Stress $= 7 - \left(\frac{4R + 6R^2}{3}\right)$ gives very good results, but is more complicated.

Mr. Stone.

(*b*) Alternating excess loads for particular wheels due to the pressure on the upper or lower guide bar, the motion of side rods, balance weights, etc.

(*c*) Lateral concussions of the wheel flanges against the side of the rail head.

(*d*) Effect of relation between wheel spacing, length of bridge panel and length of rail.

(*e*) Irregularities, local weakness, and other imperfections in the track.

(*f*) Deflection of the main girders (and also, in some degree, of the cross-girders and rail-bearers) in relation to the curve of trajectory of the engine.

(*g*) Wave period of the bridge in relation to the alternation period of the engine.

(*h*) Excess stresses induced by certain members not doing their proper share of work. Local bending or twisting stresses at joints and other stresses caused by imperfections in design, construction or workmanship.

(*i*) Irregular expansion or contraction due to changes of temperature having special local effects.

All these factors and others which might be specified, are modified in different degrees by the speed at which the locomotive travels, the design and dimensions of the bridge, the position of the member in the bridge, and the design and dimensions of the engine. Each factor is, moreover, independently variable, and in its variations affects most of the other factors to an extent which is not determinable by any mathematical process, causing in the other factors enhanced effects in some cases and diminished effects in others.

It is evident that for each particular case having its own definite conditions, the problem might be dealt with by two different methods, *viz.:*

(*a*) The analytical method. Under which each separate factor, so far as could be known or ascertained, would be examined and valued, and the net result obtained by a summation of these values*—and

(*b*) The experimental method. Under which the net result of all the stresses acting on the bar, due to all causes whatsoever, whether known or unknown, would be obtained at once as a total by means of a strain indicator.

The experimental method for obtaining the net immediate effect of the Moving Load is analogous to the use of a dynamometer for ascertaining the net tractive force required to haul a train. All elements of train resistance, whether known or unknown, are automatically

* Probably the best example of the analytical method is the paper by Professor Melan published in the *Journal* of the Austrian Engineers and Architects' Association in 1893. An abstract of this interesting paper is given in the *Proceedings* of the Institution of Civil Engineers (London), 1893-94, Vol. cxvii, p. 411.

summed up under the net total exhibited by the dynamometer. The experimental method may not be theoretically accurate, but most engineers will agree that for practical purposes it is sufficiently satisfactory. Mr. Stone.

On the experimental method, when a sufficiently wide range of experiments had been made, a series of coefficients would be obtained representing (for the class of structure dealt with), the enhanced effect of Moving Load as compared with the same weight as Fixed Load under varying conditions. The coefficients thus obtained would be used as the basis for a formula or diagram for future use in the determination of the Safe Working Stress or Equivalent Effective Stress on similar structures.

As regards the nature of experiments required, the author would invite attention to the remarks made by Mr. J. A. L. Waddell in 1892,* and also in the present discussion. The author entirely agrees with Mr. Waddell, and hopes that before long a really good series of experiments in the direction indicated may be undertaken by skilled observers on a properly organized system.

When this has been done, and the results analyzed and compared, it may be found advisable to adopt a different series of coefficients for each different design of bridge, or for each different class of members; and any desired degree of refinement in rules or formulas may then be adopted. Meantime the author ventures to hope that the average results already obtained and published in this paper may be found of use to the profession as indicating approximately the general conditions and the nature of the effects to be provided for, and as providing a sufficiently good basis for an approximate rule or formula which may be elaborated and improved when better data become available.

* "Discussion on Railway Bridge Designing," *Transactions*, Am. Soc. C. E., 1892, Vol. xxvi, pp. 276 and 277.

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No. 851.

DRY DOCKS—STONE vs. WOOD.

By Messrs. W. L. CATHCART, GEORGE W. MELVILLE, M. T. ENDICOTT, R. G. PACKARD, P. C. ASSERSON, J. G. TAIT, JOHN KENNEDY, JAMES RITCHIE, FRANCIS COLLINGWOOD, CHARLES H. HASWELL, FRANCIS T. BOWLES, ALFRED P. BOLLER and EDWARD MOHUN.

AN INFORMAL DISCUSSION AT A MEETING OF THE SOCIETY ON FEBRUARY 10TH, 1899.

Mr. Cathcart. WILLIAM LEDYARD CATHCART, Esq.—The modern war vessel is practically helpless without the dry dock, in which she must be placed periodically for the cleansing of the hull, and, with more or less frequency, for the examination and repair of all exterior parts below the water line. The expansion of the fleet and that of the system of national dry docks should, then, go hand in hand; and yet, with a rehabilitated navy, which is already formidable in strength, the United States is far behind the other great powers of the world, not only in the number, but apparently in the material, of its dry docks. Of the eleven which this Government now controls and operates, only four are of masonry, the remainder being timber constructions; and it is proposed further, in recent legislation, to build three more docks of wood.

Practice of Foreign Governments.—The practice of foreign governments differs widely from this. In Europe, not only national graving docks, but all great modern works of this kind are constructed of stone or concrete masonry. Thus, the British Government owns or

controls sixty-two graving docks—some of which are locks, which answer the purposes of dry docks—and only one of these is of timber, that at St. John's, Newfoundland, which was built by the Colonial Government. France has thirty-six, Germany has eleven, and Italy has twelve, graving docks, all of which, in the three countries, are of masonry. In Germany, a large dry dock was recently designed for a corporation there, which was to be subsidized by the Government. When, however, the dock company proposed to use timber as the material, the Government declined to aid, and it was only upon condition that the dock be of masonry that the subsidy was finally granted. Mr. Cathcart.

It will be seen, then, that the United States Government is the only one in the world which has constructed, directly, a timber dry dock. The governments of Europe have considered it the policy of wisdom to build these important works of the most enduring materials, without regard to cost. This decision has been reached, doubtless, in view, not only of the natural decay, the frequent repairs and the large expense thereof, to timber dry docks, but, as well, because of the superior stability, permanence and safety of a dock which is built of stone or concrete.

Timber vs. *Stone in Dry Dock Construction, with Regard to Deterioration.*—Timber dry docks are subject to decay, especially in the upper portions of the works, which are alternately wet and dry. The deterioration is especially swift in a warm, moist climate; but, even in northern latitudes, where decay is much slower, docks built of wood require, in about 25 years, very extensive renewals. In southern waters, submerged timber, also, perishes rapidly from the attacks of the teredo.

On the other hand, the masonry dock, if built of granite or other enduring stone and mortar of the best Portland cement, is a permanent work. In the older stone dry docks of the United States, where the mortar was made with native hydraulic cements, the masonry has suffered some injury to the joints of the entrance works, from which there was resultant leakage. This has occurred to a small extent, with the docks at Boston and Norfolk; but there has been no serious dislocation of the masonry, and the small amount of leakage has injured the structure in no way and has involved little expense for renewal. These two docks, although sixty-five years old, have had inconsiderable repairs to the masonry. The same action, from the same cause, took place with the stone dock at the Brooklyn Navy Yard. In this case, owing to the strong pressure from springs upon the site, the disintegration of joints by frost was considerable, and there was heavy leakage. Repairs, made in the best manner, were, however, wholly effectual. As a general principle, it may be broadly stated that it is possible to make a masonry dry dock absolutely water tight.

Mr. Cathcart.

Cost of Maintenance of the Various Stone and Timber Dry Docks of the United States Navy.—The relative values of stone and timber in dry-dock construction may be considered, first, with regard to the cost of maintenance of the docks, and, secondly, with regard to their stability and safety. The first consideration only will be discussed herein.

As to this maintenance, it is obvious that the proper basis of comparison is the relative costs of repairs to the structures proper, not including the accessories, such as pumping plant, caisson, floating gates, etc., inasmuch as the cost of maintenance of the latter should be the same in all similar docks, without regard to the material of construction of the dock proper. The history of the national dry docks of the United States is instructive in this respect.

Stone Dry Dock at the Navy Yard, Boston.—This dock was completed in March, 1834. The record of its repair and maintenance prior to 1866 is incomplete and not readily accessible. From that time, however, to September 1st, 1897, a period of 31 years, the expenditures for these items were as follows:

Repair and maintenance of dock proper		$62 000
Pumping plant	$53 764	
Caisson and swinging gate	14 899	
		68 663
Total		$130 663
Average, for dry dock proper, per year		2 000

It will be observed that less than half of the total amount, or 47%, was expended upon the dry dock proper.

Stone Dry Dock at Norfolk, Va.—This dock was built of cut granite, upon the same plans as the dock at Boston. The early records as to its maintenance are not available. From 1882 to 1897, a period of 15 years, the expenditures were as follows:

Repair and maintenance of dock proper		$18 776
Pumping plant	$12 108	
Caissons or floating gates	30 546	
		42 654
Total		$61 430
Average, for dry dock proper, per year		1 251

It will be noted that the expenditures upon the dock proper are but 30% of the total.

Stone Dry Dock at Brooklyn, N. Y.—This dock was completed in 1851. The figures for this dock do not compare favorably with those already given, owing to the character of the soil in which it was con-

structed and to the quality of the cement used in the joints. Many difficulties were encountered with the foundations, arising from the large springs upon the site, which were never driven entirely from the locality. The stratum through which the springs flowed was 30 ft. below the foundations. As the water was entirely fresh, as its volume was not affected by the rise and fall of the tides, and as the temperature differed from that of the water in the bay, it is unquestionable that the flow came from a source much higher than tide-water. Mr. Cathcart.

Again, the mortar used in the construction of this dock was not made from the fine Portland cement manufactured at the present day, but from the hydraulic cements which were used in all structures of its type at the time of its building.

The heavy pressure from the springs referred to, combined with the inferior quality of the cement, and the action of frost, tended to produce disintegration of the masonry, to some extent, with, in consequence, very considerable leakage, thus necessitating during the fiscal years 1886–87 and 1887–88, extensive repairs, amounting to about $88 000. It became essential to take up and relay most of the masonry at the entrance and a small portion at the head. The repairs, made with great care, and with the best Portland cements, were wholly successful, and, during the 10 years which have passed since their completion, there have been no signs of failure of the masonry, to any extent.

Despite these adverse conditions, the total cost of repairs and maintenance of the dock proper, for 45 years ending with 1897, has been but $210 618, or an average of $4 680 per year. The cost of repairs to the caisson, gates, etc., during the same period, was $69 762. The cost of maintenance of the pumping plant during this time is not available.

Stone Dry Dock at Mare Island, Cal.—This dock was completed in 1891. Since that time, up to, and including 1897, the expenditures have been as follows:

Repair and maintenance of dock proper		$1 613
Pumping plant	$8 258	
Caisson or floating gate	4 534	
		12 792
Total		$14 405
Average, for dry dock proper, per year, during this period of 7 years		$230

The expenditures upon the dock proper were, in this case, but 11% of the total. It will be observed that the figures given do not include the cost of a 40-ton crane and track, and of a spare caisson which was

Mr. Cathcart. provided as a relief in case of repairs to the original. In some published comparisons, to be later referred to, a large proportion of these items would seem to have been charged erroneously against repairs to the dry dock proper.

Similar expenditures upon the wooden dry docks of the Navy will now be considered.

Timber Dry Dock No. 2, Navy Yard, N. Y.—This dock was completed in 1890. The expenditures upon it for the first 7 years of its life were as follows:

Repair and maintenance of dock proper		$1 982
Pumping plant	$3 086	
Caisson, gates, etc	12 706	
		15 792
Total		$17 774

This would give, during this period, an average cost of repairs per year, to the dock proper, of but $283; but the story does not end here. The timber work of this dock has deteriorated considerably, and an inward movement of the abutments has taken place. The dock, in consequence of these defects, was surveyed by a board of civil engineers in 1898, and its condition was such that they recommended an entire remodeling and reconstruction of the entrance in concrete instead of timber. At its last session, Congress appropriated the sum of $300 000 for making these repairs.

If this work be completed in the year 1900, the dock will have cost, for repairs and maintenance, during the first 10 years of its life, $317 774; and, for the dry dock structure proper, $301 982; or an average of $30 198 per year.

It is quite true that this large sum for repairs in concrete is much more than would be the cost of the work in timber; and yet the Navy Department has considered the expenditure necessary in order to make the dock wholly trustworthy.

Timber Dry Dock at the Navy Yard, League Island.—This dock was completed in 1891. During its first 6 years of life, the expenditures upon it were as follows:

Repair and maintenance of dock proper		$10 202
Pumping plant	$3 632	
Caisson	113	
		3 745
Total		$13 947
Average, for dry dock proper, per year		$1 700

Mr. Cathcart.

The upper portion of the timber work, however, has decayed, to a very great extent, and there has been an inward movement of the abutments which has threatened an increase sufficient to prevent the seating of the caisson in its groove. In view of these facts, Congress, at its last session, appropriated $64 000 for repairs. If these repairs be completed during the present year, the dock will have cost, within the first 9 years of its life, $77 947; and the repairs and maintenance of the structure proper, $74 202, an average of $8 245 per year.

Timber Dry Dock No. 3, Navy Yard, N. Y.—This dock was completed in the spring of 1897. In the 2 years of its existence, it has cost $171 000 for repairs, an average of $85 500 per year.

These extraordinary expenditures, it is but just to say, resulted from a failure of the dock, due to faults in the construction, not in the design. If the work had been faithfully carried out, the injuries would not have occurred, and the expenditures for repairs, during these 2 years, would probably have been merely nominal. On the other hand, the failure was of a character which could not have occurred in a masonry or concrete dock.

Timber Dry Dock, Navy Yard, Norfolk, Va.—This dock was completed in 1889. The expenditures upon it to June, 1897, or during the first 8 years of its existence, were as follows:

Repair and maintenance of dock proper		$13 576
Pumping plant	$8 312	
Caisson or floating gate	1 532	
		9 844
Total		$23 420
Average, for dry dock proper, per year		$1 697

Since the date given above, however, Congress has made two appropriations for repairs to the structure, aggregating $35 000. These will have been expended within a few months, making the total cost for repairs to the dry dock structure proper during its 10 years of life $48 576, an average of $4 857 per year.

Timber Dry Dock at Port Royal, S. C.—This dock was completed in 1895. For the first 2 years the repairs upon the dry dock structure proper amounted to $4 262, an average of $2 131 per year.

Since this period, however, about $12 000 have been expended in repairs to the structure, making for 4 years a total of $16 262, or an average of $4 065 per year.

In summing up, finally, the relative cost of maintenance of the stone and wooden docks of the navy, the stone docks at Boston, Norfolk and Mare Island will be considered and that at New York omitted, because, from the character of its foundations and the cement used originally in its construction, it is not fairly representative.

Mr. Cathcart. From the wooden docks there will be omitted also Timber Dry Dock No. 3, at New York, the large expenditures for which are due to faults in building; but the timber docks at New York, League Island, Norfolk and Port Royal will be included (see Table No. 1).

It will be seen that the dates previously set forth herein, cover for the three stone docks, an average period of $17\frac{2}{3}$ years, and for the four timber docks an average period of but $8\frac{1}{4}$ years, which period is, however, as long as the age of the latter permits. The total expenditure on the structures proper has been, for the three stone docks, $82 389, or an average of $27 463 per dock during the average period of $17\frac{2}{3}$ years. The corresponding cost of the four timber docks has been $441 022, or an average of $110 255 per dock during their average life of $8\frac{1}{4}$ years. The average per dock structure proper per year on the basis of the figures given above is, for the stone docks, $1 558; for those of timber, $13 364, or more than eight and one-half times as much. Results such as these would seem to make most questionable the further use of timber in the building of national docks.

Disregarding, however, all comparisons, the fact remains that the four timber docks referred to have cost in repairs to the structures proper during an average life of $8\frac{1}{4}$ years in expenditures and sums appropriated the large amount of $441 022. As has been said, the failure of Timber Dry Dock No. 3, at New York, was of a character which could not have occurred in a masonry or concrete dock. There is ground, therefore, for adding the cost of repairs upon its structure to the above amount, which will give a grand total of $612 022 expended in repairs, during a comparatively brief period, to the timber docks proper of the navy.

Published Comparisons of the Cost of Maintenance of Stone and Timber Dry Docks.—Efforts have been made from time to time in testimony before Congressional Committees, in the public press and in pamphlet literature, to show that the cost of maintenance of dry docks of timber is much less than that of docks of stone. In 1897 the Simpson Dry Dock Company, of New York, issued such a pamphlet, which, since it is a public argument for timber and against masonry as the material of national and other dry docks, may be properly discussed herein. The pamphlet states that "the estimated cost of repairs to all of the docks is based upon close observations and our experience of over forty years." Estimates, it may be said, are not facts; and it would appear that the absence of exact knowledge as to the cost of maintenance of the various national docks has led, in this pamphlet, to deceptive conclusions. It should, as well, be repeated emphatically here that a just comparison of the maintenance of the two classes of docks can be based only on the cost of repairs to the dock structure proper, excluding all accessories, such as caissons and pumping plants.

Mr. Cathcart.

The Simpson pamphlet estimates the total cost of repairs to the Stone Dry Dock at the Navy Yard, New York, from its completion to 1897, as $250 000, an average of $5 434 per year. Excluding accessory structures, the actual figures, as already given, are $210 618, or $4 680 per year.

Again, there is given a total of $16 000 for general repairs to Timber Dry Dock No. 2, at the Navy Yard, New York, an average of $923 per year. The facts are that the repairs to the dock structure proper were only $1 982, or $270 per year; but, as already shown, the cost of repairs, renewals and reconstruction of the entrance will be $300 000, which will bring the yearly average for repairs to the dock-structure proper up to $30 198. It should be stated, however, that when this work is completed, no further repairs will, probably, be required for a considerable time.

The Simpson figure for general repairs to the Granite Dry Dock at Norfolk since the year 1865, is $125 000, or $3 906 per year. It will be observed that this is not claimed as an actual, but as an estimated, expenditure. In the Government report, of the Board on Dry Docks, of 1898, the total repairs from 1882 to 1897, only, are given. It is therefore to be presumed that there are no data available as to expenditures during prior years. This report gives a total outlay, for these 15 years, of $61 430, of which only $18 776 were applied to the structure proper, a yearly average of but $1 251. The Simpson estimate is from the year 1865, and it is a matter of history that this dock and its gate were seriously injured by the Confederates during the Civil War. Large amounts were spent in repairing the damage caused by their attempt to blow up the dock, and, even if the Simpson estimate be received as correct, it is apparent that the bulk of the expenditures must have been unusual and not due to ordinary wear and tear.

With regard to the Timber Dry Dock at the Navy Yard, Norfolk, the Simpson Company estimate the repairs to the dock structure, up to the year 1897, at $6 000, or an average of $818 per year. The expenditures given by the Government Dry Dock Board must, however, be regarded as authentic. These show, on analysis, that during this period the repairs to the structure proper of this dock amounted to $13 576, or an average yearly cost of $1 697, and when, in the current year, the repairs appropriated for and now being made are completed, the average annual expenditure will have been $4 857 upon the dry dock structure proper.

The Granite Dry Dock at the Boston Navy Yard is estimated, in the Simpson pamphlet, to have cost for general repairs, from its completion to 1897, $150 000, or a yearly average of $2 343. The Dry Dock Board gives the total expenditures from 1866 to 1897 as about $131 000, of which one-half was upon the dry dock structure proper, making an average yearly expenditure upon the latter of about $2 000.

Mr. Cathcart. No precise record seems to have been available giving the cost of repairs in detail prior to 1866; but the Board states that during the years 1855 and 1857 about $21 000 were expended upon new engines. It is more than probable that the repairs to the dry dock structure proper, from the time of its completion down to 1866, were very slight, since the annual report of the Chief of the Bureau of Yards and Docks for the year 1853 states that the stone dry dock at Norfolk had then been in use for twenty years, and was in perfect condition.

It would appear, therefore, that at least $21 000 of the general repairs, estimated in the Simpson pamphlet at $150 000, must have been applied to the pumping plant prior to 1866, and, if this estimated total of general repairs is correct, the average expenditures upon the dry dock structure proper, from its erection to the year 1897, must have been very much less than $2 000 per year, probably about half that amount, since the Dry Dock Board gives about $65 500 as the expenditures upon accessories from 1866 to 1897, and a further amount of $21 000 as spent upon new engines in 1855 to 1857. These two items make a total of $86 500, which leaves but $63 500 of the Simpson estimate for the maintenance of the structure proper during about 65 years, an average of less than $1 000 per year.

In referring to the concrete and granite dry dock at Mare Island, Cal., the Simpson Company began their estimate of repairs from the year 1891. From this time to 1896, they give the general repairs as $25 000; to this they add $50 000 for a new caisson—which was a duplicate and not a replacement—obtaining thus, for general repairs, a most erroneous total of $75 000, with an absurd yearly average of $12 500. As a secondary proposition, they state, that, if the item of the caisson be omitted, the repairs will average $4 166 per year, and add, finally, "this is claimed to be a modern, stone dry dock in every respect."

These estimates will now be studied in the light of the facts. A statement, as to the actual expenditures upon this structure and its accessories, obtained upon inquiry, gives the total for repairs, from the dock's completion in 1891 to 1897 inclusive, as only $14 406.86, of which amount $8 258.55 were upon the pumping plant, $4 534.59 upon the caisson, etc., and but $1 613.72 upon the dock proper, an average of $230.53 per year during this period of 7 years. The average expenditure for general repairs was $2 058.12, which figure is just half of the corresponding Simpson estimate.

The cost of the caisson mentioned has no bearing either on facts or estimates as to repairs. It is a duplicate, built as a relief for that within the dock, and its cost, $50 000, belongs to equipment, not to repairs. There is no place at or near Mare Island where an injured caisson could be docked; and, in view of this isolated situation, the Navy Department decided to have the duplicate built.

The erroneous estimate of $25 000 for general repairs, as given above, can be accounted for only by the supposition that there were included therein certain expenditures made in 1896 for a 40-ton crane and track for use about the dock, which, as with the duplicate caisson, must be classed as equipment, not repairs. Mr. Cathcart.

It will be seen, therefore, that the repairs to this "modern stone dock" were practically insignificant, in respect of the structure proper and exclusive of the accessories, which are of a class belonging to all dry docks.

The Simpson statement as to the timber dock at the Navy Yard, League Island, gives an amount of $10 000 for repairs and improvements from March 4th, 1891, to January, 1897, a period of about 6 years. By deducting $4 000 for raising the coping and $5 000 for repairs caused by the method of carrying out the work upon an adjacent wharf, the cost of maintenance of the dock is reduced to a total of $1 000, and an average of $166.67 per year.

As has been shown previously, Congress has appropriated $64 000 for repairs to be made during the present year to the structure proper of this dock. It is scarcely necessary, therefore, to challenge the estimate as above, since this appropriation alone gives a yearly average of over $7 000 for repairs to the dock proper.

The timber dry dock at Port Royal, S. C., was not built by the Simpson Dry Dock Company. Their pamphlet gives an estimate for general repairs to dock and accessories of $5 000 for the 2 years, or $2 500 per year. This yearly average is believed to be substantially correct.

In concluding the analysis of the cost of maintenance of the various national dry docks, as estimated in the pamphlet issued by the Simpson Dry Dock Company, it is desirable to state that this analysis has been undertaken only in view of the fact that this company is foremost among the advocates of timber dry docks, and that its arguments, as published in this pamphlet, and as presented to the Naval Committees of the two Houses, have had much influence in shaping the action of Congress with regard to the material of construction for the national dry docks.

In considering the cost of repairs to timber dry docks, as given in this pamphlet and in other arguments presented by the advocates of modern docks, it will be noted that the timber docks of the Navy, which are cited as examples of the small expense of maintenance for this material, range in age from but 2 to 10 years. Being thus practically new, it is obvious that their deterioration cannot have progressed sufficiently to give a true indication of the cost of repairs in the long run. For a correct estimate as to the latter, the comparison must extend over a period, at the close of which decay shall have reached such a stage that it will be necessary to make repairs sufficiently extensive to

Mr. Cathcart. restore the dock to its original sound condition. In other words, make the comparison, not on repairs for temporary patch-work, but on the reconstruction of a fair portion of such a dock, built, as it is, of perishable material.

Even such comparison as it has been possible to give herein, from the data of these practically new timber docks of the Navy, will show, however, that their yearly cost of maintenance, as presented by the advocates of wooden structures, is misleading and far below a correct measure of this item. These docks are certainly equal, if not superior, to the average of timber dry docks constructed elsewhere. It is not possible to admit that their care is in incompetent hands; and yet, it is clearly apparent that their cost of maintenance is very large, exceeding greatly that given in the pamphlet referred to and in other reports.

The comparison instituted herein has been based upon the cost of repairs to the dry dock structure proper. The advocates of timber construction seem to prefer a comparison founded on the entire general repairs of the docks, including all accessories, such as caisson, pumping plant, etc. It is submitted, that, even if the latter method were proper, the showing which has been presented to the public is unfair, in that the timber docks, being from but 2 to 10 years old, have not reached a time when these accessory structures require renewals or extensive repairs. To obtain correctly this yearly average for general repairs, which timber advocates prefer, this stage of renewals to accessories must be awaited. When it shall have been reached, it is plain that the comparison will be much more unfavorable to the wooden dock.

With regard to the stone docks of the Navy, it must be said, also, that the cost of their maintenance, as presented by their opponents, is largely misleading. The comparisons given, as to these docks, by timber advocates, are founded, not upon the maintenance of the structure proper, but upon the general repairs to the dock and its entire accessories—a principle which is obviously wrong. The errors arising from the adoption of this method are magnified in this case by the very considerable age of the majority of these four docks, two of them being 65, and one 48 years old. The repairs to the accessories form, therefore, a very large proportion of the total cost of maintenance, amounting, with the dock at Norfolk, for 15 years, to 70% of the total; with that at Boston, for 31 years, to 53%; and with that at Mare Island, for 7 years, to 89 per cent. The maintenance of the timber docks, on the other hand, if including repairs of all kinds, has not, because of their youth, comprised any considerable expenditures upon caissons, gates or pumping machinery.

In summation, it may be said that the data set forth herein would appear to show conclusively that the average yearly expenditure for the repair and maintenance of a masonry dock, built in the best manner

and of superior materials, is inconsiderable, both in the aggregate and in the percentage of the original cost; while the same items, with regard to the timber docks of the Navy, are unquestionably very large. Mr. Cathcart.

In view of the fact, that, for 10 years past, the Government has been practically committed to the policy of timber docks, having, within that period, built six of this material, and, of the further fact, that, by recent legislation, three more of these structures are authorized, the subject is presented to the members of this Society for discussion. The growth of the United States as a nation, and the results of the recent war, foreshadow an early and considerable increase in the force afloat. The defense of a coast line, extraordinary in extent; the acquirement of territory beyond the sea, which must be held—as it was won—largely by the Navy; the probable entry of the United States into Asiatic politics; and the prospective revival of the merchant marine, which the fleet must lead and guard—all point to this. The full consideration of any factor, therefore, which is essential to naval strength—as a trustworthy system of dry docks surely is—must be of interest, not only to the engineer, but to every man who has at heart the welfare of the country's gallant fleet.

TABLE No. 1.

Dock.	Years.	Total on dock proper.	Average, dock proper per year.	Previous average per year.
Stone, Boston	31	$62 000	.....	$2 000
Stone, Norfolk	15	18 776	.....	1 251
Stone, Mare Island	7	1 613	.....	230
Totals	53	$82 389	.....	$3 481
Average	17¾	27 463	$1 558*	1 160
Timber, No. 2, New York	10	$301 982	.....	$30 198
Timber, League Island	9	74 202	.....	8 245
Timber, Norfolk	10	48 576	.....	4 857
Timber, Port Royal	4	16 262	.....	4 065
Totals	33	$441 022	.....	$47 365
Averages	8¼	110 255	$13 364*	11 841

* The item $1 558 is on the basis of three docks at 17¾ years per dock. Similarly, the item $13 364 is based on four docks at 8¼ years per dock; $13 364 is about 8¼ times $1 558; or, taking previous averages, $11 841 is about 10 times $1 160.

Commodore GEORGE W. MELVILLE.*—This subject, in the speaker's opinion, is of prime importance to the further development and maintenance of that strength upon the sea, of which—more and more as the years pass—the nation will find itself in need. Com. Melville.

* Chief of Bureau of Steam Engineering, U. S. N.

Com. Melville. With the acquirement of distant island territory, swarming with bronze-hued aliens, "The White Man's Burden," of which Kipling writes, in the civilization and enlightenment of "the silent, sullen peoples," is laid at last on the broad shoulders of the United States. With this there looms up as well a golden future on the Pacific, in the extension of the nation's trade to its uttermost island and to the vast Asian littoral beyond. Finally, the time seems ripe for the merchant marine to take again its proud place as of old, when its flag streamed over every sea.

These considerations point with no uncertain hand to the nation's need in the early future of a strong force afloat; and, in maintaining the efficiency of that force, a worthy system of dock construction is a cardinal essential. The speaker desires, therefore, to place on record his opinion—as he has done in the past—with that of the Secretary of the Navy, with that of his distinguished colleague, Commodore Endicott, and with that of his predecessor, Admiral Matthews, as being opposed most strongly to the flimsy structures, costly in their weakness, which have been built as timber dry docks for the Navy. As to the use of wood for the docks of private corporations, the speaker has nothing to say. It is plentiful and cheap; the average draught of merchantmen is light; the structures are less costly; and the exigencies of private business may require them. With Government constructions, however, none of these considerations holds. A wise nation builds, not only for its generation, but for its sons of the coming years; and we, in our day, are not true to this principle in the timber-veneered excavations, perishable and perilous, which, for 10 years past, have been inflicted upon the Navy.

The speaker will consider, first, the comparisons which have been advanced by the opponents of stone docks, with regard to the first cost of the two types. In this matter it has been the practice of some timber advocates to compare the cost of wooden docks, built by contract for the Government, with that of granite docks constructed in the navy yards by day's labor, extending over a long series of years, as appropriations were made from time to time by Congress. Thus, it is found that the Simpson Dry Dock Company, in its public arguments, has instituted such comparisons as to first cost between its own timber docks, built by contract for the navy in from 22 months to 2½ years, with stone docks the construction of which by the Government extended, respectively, over 6, 10 and 19 years.

Such comparisons are grossly unfair. The only just system as to this would be to compare timber docks and stone docks built under exactly the same conditions; that is, by contract, after public advertisement, under authority from Congress to enter into contract for the full completion of the docks at the earliest possible date. For the first time in the Navy's history it is now possible to compare the relative

cost of timber and masonry docks of approximately the same size and to be built under substantially the same conditions. Com. Melville

The proposals opened recently for a dry dock at Boston will furnish a fair comparison in this respect. The lowest bids for this structure show that the Government will be able to build by contract a dock of the very largest size, constructed of concrete and stone masonry, within the sum of $1 000 000, or at an advance of but 25% upon the amount appropriated by Congress for each of the three timber docks now authorized. In view of the cheapness of wood, as compared with concrete and stone, it would appear either that its manufacture into a timber dock is a somewhat costly operation, or else that there is a very large profit included in the sum appropriated by Congress for these timber docks. The matters of first cost and of cost of maintenance, however, are minor issues only, which should have no determinate bearing.

The real question is the stability and safety of a structure which may, at any time, hold within it a great battleship, costing millions of dollars and swarming with hundreds of men. The nation, to save dollars, cannot afford to imperil its naval strength or to risk human life; and these very things its policy of unwisdom as to docks compels daily in the entrance of ships into structures which have shown, time and again, their weakness. None of the timber docks of the Navy exceeds 10 years in age, and yet abutments have moved inward, and other injuries, more or less serious, have occurred to them, extending from the bursting in of the altar system, as at League Island, Port Royal and New York, to the distortion of the floor of the dock, as at Port Royal, and the partial collapse, in the case of Dry Dock No. 3 at New York. That no catastrophe—great in financial loss and appalling in the destruction of life—has yet occurred, is due solely to the care and skill of those in charge of these weak docks. The battleship *Massachusetts* was saved—during the partial collapse of Timber Dock No. 3 at New York—only by the hasty flooding of the structure and the hurried removal of the ship from its treacherous grasp.

In discussing the relative strength of the two types of docks, the opponents of stone lay stress only upon the capacity to withstand the vertical thrust of the ship upon the flooring of the dock. The stone dock may rest on a bed of natural rock, or in one of impervious clay, or upon piling foundations. The timber dock is built always upon piles. The calculations referred to, therefore, in the most difficult cases, consist simply of those based upon the strength, penetration and cross-section of the piling, involving but little engineering skill. These statements are *ex parte* and fallacious, and ignore wholly the inherent and vital weakness of the timber dock, which lies in its tendency to inward collapse under the hydrostatic pressure horizontally

Com. Melville. and from without, upon its sides, and vertically from beneath, upon its floor.

No such danger threatens a properly constructed dry dock of masonry. With good materials and workmanship, it is possible to make it absolutely water-tight, and the structure has, in itself, a weight more than sufficient to withstand the hydrostatic pressure from without. Stone and concrete give it that weight, and, since these materials, when properly embodied, will last for centuries, that weight can never be lost. For all practical purposes, the stone dock is as enduring as the everlasting hills.

With the timber dock the case is radically different. Its integrity —fleeting, at best—depends solely upon success in pinning it to the ground. Such a dock has merely a thin veneering of timber, lining the excavation and held down by piles driven into the soil, to which piles the timbers are bolted. It would be, obviously, both futile and dangerous to attempt to exclude from such a structure the entrained water in the surrounding soil. The timber dock, therefore, is one of tight joints only in the abutments, being open as a sieve elsewhere, and leaking, more or less, as the amount of water in the circumjacent ground is large or small. It lacks utterly, therefore, the tightness and weight which give stability and permanence to the dock of stone.

Furthermore, it is apparent that this hydrostatic pressure—which, in graving docks, is the chief force to be met and guarded against—increases with the depth. As this dimension increases, therefore, the more unfavorable become the conditions for the wooden dock, which has relatively no inherent weight to oppose resistance to pressure from without. With such increase, its stability and safety suffer rapid decrease, and there is swift growth in the tendency to rupture the bottom and sides. From this point of view, the question becomes one of great importance and risk, with regard to the very deep docks which are now necessary for the larger cruisers and battleships of the Navy.

It has been shown that the leakage of the timber dock depends simply upon the amount of water present in the adjacent soil. Now, this very leakage, which in one sense is an element of safety to the structure, is, in another, a source, ultimately, of sure and grave danger; for soil is carried in with the leakage, the latter increases, and there is displaced finally so much material as to cause at least a partial failure. This was the case recently in Dry Dock No. 3, at the Navy Yard, New York.

In a stone dock, however, a leak is not followed by such results, since the structure is homogeneous and all joints are water tight. The failure of one or more of the latter does not, therefore, involve necessarily the failure of others. As a matter of fact, the stone docks at the Navy Yards of Boston and New York have leaks to-day which have

existed for many years without measurable increase and without involving the structures to any extent. It is true that the masonry at the entrance of the Boston dock has been displaced slightly by the action of frost upon the open joints; but the weight of the structure is such that, if these joints were all open, it would still act as a dry retaining wall or arch, without danger of failure. Com. Melville.

At least two of the timber dry docks of the Navy suffer from excessive leakage, with its resultant dangers. At the Norfolk yard, the wooden dock is founded in a bed of clay of excellent quality, and the leakage is very small. That of Timber Dock No. 2, at the Navy Yard, New York, amounts to little more than 100 galls. per minute, which is moderate, considering the character of the soil. The leakage of the timber dock at Puget Sound, is not much more; but this is due chiefly to the fact, that, in the construction, provision was made for admitting the water from the soil, through a system of sub-drains and valves, into the drainage culvert, and thence to the pump-well, from which it is discharged by the drainage pumps. This was done in order to relieve, largely, the bottom of the dock from hydrostatic pressure.

With the wooden dock at League Island, however, no such advantageous conditions prevail. It has always leaked excessively, the amount entering from the surrounding soil varying from 1 200 to 1 700 galls. per minute. This great volume taxes fully the drainage apparatus originally installed; and, to obviate the inconvenience and risk which would arise from the failure of this pump, a duplicate apparatus has been erected. This duplicate pump stands to-day an eloquent, if silent, monitor as to the unwisdom of the wooden dock for national use. When a ship is in the dock and it is necessary to keep the floor dry, one of these drainage pumps runs constantly.

At the Port Royal Dry Dock, the leakage is about the same, and it was found necessary to fit reliefs very largely in the bottom and sides, in order to prevent a recurrence of the upheaval and rupture of the structure, which occurred about the time of its completion.

The foregoing has referred to the stability and safety of national docks of wood, in the past. The speaker will now examine briefly the programme for the future. In the Act of Congress, approved May 4th, 1898, there were authorized four new dry docks for the Navy, one each at Portsmouth, Boston, League Island and Mare Island—all to be of the largest size. All these docks were to be of timber, at a cost not exceeding $825 000 each; but the Secretary of the Navy was authorized, in his discretion, to build one of granite, or concrete faced with granite, the limit of cost, in this case, being increased by $200 000.

In the exercise of the discretionary powers lodged in him, the Secretary of the Navy decided to build the dock at Boston of granite, or concrete faced with granite. As stated previously, the bids for this

Com. Melville. dock fall considerably within the appropriation and are about 25% in excess of the amount appropriated for each of the wooden docks.

The absurdity of a further continuance of the policy of extravagance and disaster in the building of wooden docks is shown, not only by the facts above given as to the relative costs of the types, but by further facts with regard to the soil in which the proposed docks are to be located. For instance, that to be built at League Island will meet the same adverse conditions to which its timber predecessor there has succumbed. The soil there contains a stratum of compact sand and gravel, which is water-bearing, and the surface-water, when it meets this stratum, flows into the dock, the latter acting as a huge drain. Shall there be erected at League Island, more duplicate pumps, as costly monuments to the timber folly?

Again, borings at the possible sites for the proposed timber dock at Portsmouth, N. H., show, thus far, that the excavation for this dock must be blasted from granite, and then, to comply with law, be lined with wood. The advocates of timber could ask no deeper devotion to their theories and interests, than this.

In the speaker's view, Congress has acted in this matter without full knowledge of all that bears upon the case. Commodore Endicott, in a report to the Secretary of the Navy, which has been transmitted to Congress, has set forth the facts with clearness and great strength. The matter is of such importance to the future of our gallant fleet, that its discussion cannot be too wide. As an engineer who has served both on land and sea, the speaker regards it, therefore, as a duty and a pleasure to present his views on this subject.

Com. Endicott. M. T. ENDICOTT,* M. Am. Soc. C. E.—It is probable that the first dry docks were constructed in Great Britain. It is known that, in the time of Queen Elizabeth, docks for the reception of vessels needing repairs were made by excavations into the soil bordering water-ways, lined with timber and backed with clay to protect the floor and sides, and fitted with gates at the entrance to shut out the water. The vessels at that time were quite small and the docks were of corresponding dimensions.

In the time of Charles II several docks were constructed in English ports, for the use of the British Navy, which were of considerable size. These were also of timber.

So far as the records now show, all the earliest docks were of wood, and were what are now called graving docks; that is, the dock was formed by making an excavation in the earth and lining the bottom and sides, thus forming a receptacle for the vessel, into which it could be floated, landed upon supports, and left accessible for examination and repairs when the water should be exhausted from the dock. As time went on, and larger and better docks were called for, the con-

* Chief of Bureau of Yards and Docks, U. S. N.

struction gradually changed, and brick and stone came into service for the construction of certain portions of the docks, and at present there are on the Thames many docks which are built of all these materials in combination; that is, partly of wood and partly of brick or stone, and, in the case of those of later construction, of concrete, also. Com. Endicott.

The transformation has progressed in Europe to such an extent that, at the present time, practically all the large docks are built exclusively of masonry, classifying concrete, for the present purpose, under the head of masonry, although strictly speaking it is not. The records, so far as accessible, show that abroad there are no constructions of this kind in timber. The present Chief Constructor of the Navy, Mr. Hichborn, who visited Europe in 1885, states that at that time the later docks were built largely of concrete, and were of the most substantial construction; and speaking of those on the Thames (private docks), a number were built of wood, but the principal docks were of masonry or concrete.

Masonry docks are the outgrowth of some centuries of experience in the construction and use of works of this kind. In foreign practice, it is a case of the survival of the fittest.

The first dry docks in America were constructed by the United States Government, and were begun about the year 1828. Before these were undertaken, Congress authorized the Secretary of the Navy to engage a skilful engineer to prepare the designs for these structures, and Colonel Loammi Baldwin, of Boston, Mass., was employed. He came to the duty well fitted for the work, having prepared himself especially by a visit to Europe for the purpose of studying structures of this class. With the experience of the Old World before him, he very wisely decided to model the structures for this Government after those which had been evolved from the experience of so many years abroad, and he made the two docks then authorized for the Navy, at Boston and Norfolk, the finest examples of fine masonry in submarine works that then existed in this country. They were regarded as among the best and most creditable public works built for the Government up to that time, and were pointed to with pride as works showing great skill in design and execution, and well adapted for the purpose of docking and repairing the vessels of the Navy.

The construction of these docks extended over several years, and they were not completed until 1834. They were built at a comparatively moderate cost, the Boston dock costing about $677 000,* and the Norfolk dock about $943 000. Both were substantially founded upon piles, were built entirely of stone, and were faced with cut granite of the finest quality and workmanship.

The granite dry dock at the Brooklyn Navy Yard, completed in 1854, followed, and, for various reasons, was a very expensive structure,

* This dock was afterward extended in length, bringing its cost, to date, up to $972 000.

Com. Endicott. costing $2 000 000; and then came the dry dock at Mare Island, Cal., which is a concrete structure lined with cut granite.

In the interval between the construction of the Boston and Norfolk docks, and that at Mare Island, the Government built several floating dry docks of wood. Two or three were of the balance type, that is, of one section, and several others were of the sectional type, and in the case of those 300 ft. or more in length, the sections numbered as many as nine. The balance docks were used in connection with shallow basins, as were the sectional docks, in order that a ship, taken up by the floating dock, could, by the sinking of the dock upon the floor of the basin, be hauled out upon a railway on shore for extensive repairs, while the dock could be used for docking vessels for slight repairs, not requiring hauling out.

It is not the purpose to discuss the merits of floating docks. This is simply mentioned as an incident in the history of the dry docks of the Navy. They had their day, but not proving so satisfactory, durable or convenient as the masonry graving dock, they were not replaced. They gradually deteriorated so much as to require extensive rebuilding. To-day only one of these docks remains in service, that at the Portsmouth, N. H., Navy Yard, and it is now safe only for vessels weighing a little more than 1 000 tons.

The timber graving docks of the present day in the United States did not originate in the Navy. They were constructed first on the Great Lakes. A timber dock built at Buffalo, N. Y., as early as 1840, was inspected by the speaker a few years ago, when he was assured by the officers of the company then owning it, that it was built not later than the year named. The speaker has been led to believe, from some investigations made since, that it was built even earlier than that time, and that the timber graving dock made its appearance in this country as early as 1837 or 1838. Others followed, and many docks of this type are now to be found in Buffalo, Cleveland, Chicago, Detroit, Port Huron and other places. While they differ in details from each other and from those built on the Atlantic coast, they are all of the same general type, in that the excavation is lined entirely with wood secured to piles driven in the bottom and upon the slopes of the sides, and faced with longitudinal timbers, forming steps or altars upon the sides.

The first timber dry dock built upon the Atlantic Coast was at Boston, Mass., in 1854, fully 14 years after the construction of a similar dock upon the lakes. It was of the same general construction, and differed only in details. Other docks upon the Atlantic Coast followed, all built by J. E. Simpson & Company, so that here this type of dock has come to be called the "Simpson" dry dock.

About the year 1886 the question of the construction of additional dry docks for the Navy became an important one, as the Navy was

building vessels of steel which required frequent docking; and, as the old sectional docks were disappearing, the need for further docking facilities was felt. Com. Endicott.

Without entering into all the reasons which led the Government to abandon the construction of masonry docks and take up with those of timber, it may be said that this was due to the advocacy of this type of dock by persons in civil life who were interested in their construction, and by a few officers inside the naval service who had championed the cause of this type.

The timber dock was presented to the attention of the Navy as possessing not only many good qualities, but many far superior to those of the stone dock. The most attractive arguments presented by the advocates of the former were its cheapness, its convenience in the way of accessibility to the floor by the gently sloping sides and low altars, its width on top, admitting light and air freely, its immunity from the disintegrating effects of frost, and its greater comfort for workmen in winter weather. A negative advantage, as urged recently by one of its strongest advocates, before the Senate Naval Committee, is that "it will last 40 years, and that is long enough for anything to last."

The arguments advanced by its advocates were successful in securing favorable action in the committees of Congress, and the first appropriations for timber dry docks were made in 1887, providing for two, one at the New York yard and one at the Norfolk yard. These were followed by one at the League Island yard, completed in 1891; one at the Port Royal Naval Station, completed in 1895; one at the Puget Sound Naval Station, with a masonry entrance, which was completed in 1896, and one in the New York Navy Yard, called Dry Dock No. 3, completed in 1897. The Government was thus fairly launched on the policy of the construction of timber dry docks, and it seemed as though it would be impossible to again secure an appropriation from Congress of an amount sufficient to build a masonry dry dock; in fact, the acts making appropriation, in almost every instance, as well as limiting to an amount only sufficient for a timber dry dock, provided specifically for one of such material.

The last Congress made provision for four timber dry docks, to be constructed, one each at Portsmouth, N. H., Boston, Mass., League Island, Pa., and Mare Island, Cal., but provided that the Secretary of the Navy might, in his discretion, build one of them of granite, or concrete lined with granite, and in such case the limit of cost was to be increased $200 000, to a total of $1 025 000. The Secretary exercised the authority granted by this act by deciding that the Boston dock should be of granite or concrete lined with granite, and plans and specifications were prepared for a dock of this character. Bids were opened on January 31st, 1899, in the Bureau of Yards and Docks, and resulted in a proposition providing for a dock with granite facing and

Com. Endicott. concrete backing, complete, for about the amount of the appropriation, or about $1 000 000. This result was much more favorable than anticipated, and showed for the first time that a masonry dock of the largest size and the best construction could be built for a sum not greatly in excess of the cost of a timber dock of the same general features and dimensions.

Prior to the final passage of the act making appropriation for these last-named docks, the Department had been before the Senate Naval Committee in advocacy of the construction of granite rather than timber docks, and the Board on Dry Docks, appointed by the Department in 1897, had reported recommendations which were, in effect, in favor of the policy of docks of masonry instead of timber. This report was laid before Congress by the Department. The Department had thus taken such steps as it might to bring to the attention of Congress its views and advice in the matter of the construction of these important structures, and the action of Congress, having been substantially adverse to this advice, the Department was not disposed to pursue the matter further with respect to the docks then authorized and provided for.

The possibilities in the way of the construction of masonry docks, as shown by the letting just referred to, were of such importance that the Bureau believed it advisable to bring them particularly to the attention of the Department, and, through it, to Congress, with a view to securing a reversal of the action already taken in the act referred to.

The subject of the advisability of constructing these works of stone rather than of timber had received much attention from engineers and from the press, and there seemed to be a wide interest in the matter, and a strong public sentiment against the course prescribed by Congress with reference to the material of which they were to be constructed. It had also become quite apparent that the general sentiment among officers of the naval service who took an interest in the matter and had paid much attention to it, was, with very few exceptions, strongly in favor of masonry docks. Altogether, it seemed to the Bureau that the result of the bids for the Boston dock indicated that the time had arrived for the Department to make another effort to save the Navy from the construction of more timber docks. In making such a move there seemed to be a prospect of success, and the Bureau addressed a letter (published in the New York *Tribune*), to the Secretary of the Navy, strongly urging the Department to make another effort with Congress to abandon the construction of timber dry docks and re-enter upon the policy, which it instituted in the early years of the century, of building for its Navy the safest and best dry docks, and which had become the policy of all other first-class naval powers.

In pursuance of this recommendation, the Department has again

brought the matter to the attention of Congress, through its naval committees, and has laid before them the Bureau's recommendations in the matter, with its earnest approval. The matter has been brought up at an opportune time, for several reasons: Because the timber docks heretofore built are beginning to show that they are temporary and unsafe structures; because the proposals for the Boston dock show that a first-class masonry dock can be constructed for a reasonable sum and at a moderate increase over the cost of a timber dock; and because the public mind is alive to the subject and strongly in favor of good, substantial, creditable structures of this kind for the Navy. Therefore, the recommendations and efforts of the Navy Department, backed by a disinterested, logical and earnest sentiment, are almost sure to bring about a change in the present policy. Com. Endicott.

A timber dock is a temporary structure, and as the chief material entering into its construction is wood, it requires very extensive renewals at an early day, varying, according to the kind of wood used and the local influences, from 20 to 25 years, when they amount to a practical rebuilding of the upper portion of the structure, and frequently its entire reconstruction. While the first cost of a timber dock is small, compared with that of a masonry dock, the rapid deterioration referred to results in considerable expenditures at the end of a few years, if the structure is kept in good condition, and if the repairs and renewals are of an extent necessary to preserve the dock in a condition as safe and good as when completed. The experiences in the Navy, thus far, with timber docks, although extending over only a few years —the dock now longest in use having been built about 10 years ago— show that the expenditures necessary to restore them to their original condition at the end of this latter period are very considerable, and appear to be several times as great as those for a stone dock of about the same dimensions. For the purposes of a fair comparison the cost of the maintenance and renewal of accessory structures, as caissons, pumping machinery, etc., should be neglected, and a parallel should be drawn entirely between the expenditures made upon the dry dock structures proper; which, in the case of one, would be stone and concrete, and in the case of the other, timber.

The records of the stone docks of the Navy, as far as they have been collated, show that, generally speaking, the repairs are very small indeed; the only instance in which any individual repair has been of considerable magnitude being in the case of the stone dock at the Brooklyn Navy Yard, where the masonry joints became injured on account of the presence of springs and the action of frost. This condition was due largely to the fact that the cement used in the mortar for the joints was native hydraulic cement, not Portland cement. The work was repaired about the year 1886, the masonry at the entrance and some at the head being taken up and relaid with Portland cement

Com. Endicott. of the first quality. Although these repairs have been completed for many years, they appear to be entirely successful, and there has been no recurrence of injury to the joints. It seems quite safe to say that if high-grade Portland cement mortar had been used originally, there would have been no such trouble, and the work would probably have lasted until the present time without any material injury.

The records of the repairs to the stone docks extend over a great many years prior to the Civil War, and the collection of exact and full data in regard to the expenditures upon them is difficult, but those which have been examined and compiled are sufficiently extensive and accurate to be a fair index of the repairs to such structures.

Without repeating here the details and figures, it is to be said that the average annual cost of repairs upon the masonry docks has been very small indeed, and has amounted to a very trifling percentage of the cost of the work.

The records of the expenditures upon the timber docks and their present physical condition are full, exact and easily obtained, and show that not only is the cost of repairs per year very much greater than for stone docks, but that the percentage of the original cost put out in repairs is several times as great as in the case of the masonry docks.

In the matter of cost, the experience of the Navy has been that in the case of the masonry docks last built, it is large; but that those first built, comprising the docks at the Boston and Norfolk Navy Yards, although of the finest quality of cut granite, did not cost as much, being in each case less than $1 000 000. The large cost of the masonry docks was due to several causes; in the first place, they were not built by contract, but by day's work by navy-yard employees, and the materials were purchased or contracted for piece-meal. The appropriations for the work were made by Congress in such a way that labor and expenditures extended over long periods of time, thus enhancing the cost and resulting in decay and injury to the coffer-dams and accessory structures, and the disintegration of the soil exposed in excavations from the action of the elements, on account of the many seasons covered by the work.

The timber docks for the Navy were all built by contract, after public advertisement, and, by the authority of Congress, contracts were made for the entire completion, thus enabling the work to be taken in hand and pushed through with the greatest possible despatch. It will be seen, therefore, that the records of the Navy, in the matter of the costs of timber and stone docks as built, afford no fair basis of comparison, and one of the great advantages which has been urged in favor of the timber dock is the smaller relative cost. It has even been claimed that the interest on the cost of a masonry dock was sufficient to build and maintain a timber dock indefinitely. This comparison

has almost invariably been drawn between a timber dock built by contract, and either the New York or Mare Island stone docks, which latter were built by day's labor, the work extending through a long term of years. Com. Endicott.

An opportunity has been afforded recently for instituting a somewhat accurate comparison of the relative cost of these two types of dry dock by the proposals received for the construction of a concrete and stone dock designed for the Boston Navy Yard. These bids were based upon conditions very similar to those which obtained in the cases of the timber dry docks; that is, Congress having authorized the entire construction of the dock within a certain limit of cost, the plans and specifications were made and competitive bids were invited, after public advertisement, the work to be completed in a period not exceeding 2½ years, and as much shorter as the contractor may desire to make it.

These bids show that a dock of the largest size can be built of concrete lined with cut granite, and complete with all accessories, at a cost of about $1 000 000, or only about 25% in excess of the amount fixed by Congress as a limit of cost for each one of the timber dry docks to be built of the same size and under similar conditions. It will be seen, and must be admitted, that such a comparison is fairer and much more accurate than that previously referred to, where the cost of the timber dock by contract was about $600 000, and the cost of the stone docks at Mare Island and New York was $2 000 000 in one instance and $2 750 000 in the other.

This is the first time in the history of dry dock construction in the Navy that such a comparison of the cost, under similar conditions, has been possible. The result is, of course, beyond dispute, and so favorable to the masonry dock, in the matter of relative cost, as to be of the greatest value and significance. It shows that the relative costs of these works heretofore claimed by the advocates of the timber dock, and their general acceptance by the public, were wrong and misleading.

The claims set up for the timber dock, based upon these comparisons, have had great influence upon the public mind and upon Congress and its committees, and have resulted in the practical adoption of this type of dry dock for the United States Navy for the last twelve years, during which time the Government has more than doubled its docking facilities by the construction of six timber docks. While other claims have been made for the timber dock as superior to the stone dock, there is no doubt that the matter of first cost has largely determined the action of the committees in Congress; but the result foreshadowed in the proposals for the Boston dock is a permanent masonry dock at an increased cost of only about 25% over a temporary dock of timber, which would require extensive rebuilding in

Com. Endicott. 20 or 25 years, and also a small annual outlay for maintenance as against a very large one.

In the long run, the timber dock is much more expensive than the masonry dock. A masonry dock built by contract, after public competition, can be built in about the same time as a timber dock, at a moderate increase of cost and a small annual outlay thereafter for its maintenance, as against a timber dock 25 or 30% cheaper in first cost with a large annual expenditure for maintenance, amounting to a practical renewal every 40 years. The result shows that a masonry dock is cheaper than one of timber.

In weighing the relative merits of the stone and timber docks, in the matter of convenience for use, it can be stated that the conditions are practically the same. A timber dock, as usually constructed, is of great width at the top, which has resulted from an effort to construct the sides of the dock at the least expense and with the least danger of deformation, and the slope generally adopted is that natural to the soil, so as to avoid any considerable thrust from the earth tending to force in the sides. The side construction consists almost invariably of timbers superimposed in such a way as to form altars or steps from the bottom to the top. This makes the entire side construction practically a stairway; and the dock, being wide at the top, admits light and air freely.

Stone dry docks, as generally constructed, are not so wide at the top, and the altars or steps are of greater height. Stairways for ingress and egress are introduced at convenient distances. It has been put forth as a great advantage of a timber dock over one of masonry, that, owing to the form, the timber dock is much more convenient for access and admits more light and ventilation, thus facilitating the work in the dock. The materials of which a masonry dock is constructed lend themselves to any form, and one can be constructed of precisely the same outline as the ordinary timber dock. The general form or outline of the masonry dock is one which is almost universally followed in the construction, and, although less broad at the top than the timber dock, such construction results in no inconvenience or disadvantage to the work carried on in the dock, else the form would long ago have been changed to one more nearly approaching that of a timber dock. In the United States, the last masonry dock constructed, that at Mare Island, approximates very closely to the cross-section of a timber dock, and is a sufficient answer to the claim of the superiority of the timber dock in this respect.

It is to be observed, in this connection, that a dock very wide at the coping level is not without its disadvantages, in that it necessitates cranes, or appliances for handling materials, with very long arms to reach the center of the dock, and this has been held by European builders to be an objection to very wide docks. As a matter of fact, it

may be stated that the masonry dock of the ordinary form admits of all the light and ventilation necessary with a ship in dock. There is nothing to obstruct except the vessel itself, which will, in this form of dock, intercept considerable light, and render the portion of the dock directly under the vessel less light than other portions. It is not known that any difficulty has ever been experienced in any masonry dock through lack of sufficient light and ventilation. The only condition in which there would be any lack of light and ventilation would be in the case where the vessel contained in the dock was of such size, as to practically fill it. Where the cross-section of the vessel approximates that of the dock, this condition could only exist amidships, as the section decreases both ways from the center of the vessel. Com. Endicott.

In the matter of accessibility, while the timber dock affords a convenient stairway at any point, a stone dock admits of the installation of stairways at any intervals deemed necessary or desirable. It can be stated, therefore, with reference to the form of the docks, that the masonry dock meets substantially all the requirements of accessibility, light and ventilation, and, in any event, admits of the construction of precisely the same form or outline as a timber dock.

In the matter of the stability of the two classes of dock, the conditions to which they are subject are practically the same. The ability to sustain the weight of the vessel is met equally by the two, generally speaking. The condition involving the most danger to a dry dock, however, is not the weight of the vessel which it receives, but the pressure of the water contained in the soil below the level of high tide, which acts normally to the exterior of the dock, tending to lift the bottom and force in the sides. It needs little explanation to show that the dock which is capable of meeting this condition more successfully is the one of masonry. The timber dock is built of a comparatively light material, wood. Its weight is a very small element of stability, and its integrity depends upon the measure of success with which the access of water through the soil to the sides of the dock is prevented, and with which the wood, which constitutes the interior lining of the excavation, is pinned or secured to the soil underneath. This is effected by driving piles into the soil, to which the interior woodwork is secured by bolts. It will be seen that such a structure is not well designed for resisting the inward hydrostatic pressure, and in all timber dry docks it is the practice, particularly in those of considerable depth, to provide valves or inlets, through which the water in the soil may find access to the interior of the dock, and be carried away through the culverts to the pump-well, and thence be discharged into the harbor. It has been well said that the only way to preserve the stability of a timber dry dock is to let the water into the dock and then pump it out. The pressure of the water is particularly felt in the entrance works of the timber dock, which, being

Com. Endicott. near the harbor, are subjected to greater hydrostatic pressure, where the slopes are much steeper, and to the water pressure upon the sides is added a great deal of thrust from the soil. It has been frequently found in the timber dry docks that they have been incapable of successfully resisting this force, and in many instances the altars have been forced off, the sides of the entrance works have been forced inward to some extent, the bottom of the dock has been uplifted, and, where defects in the work have existed, an inflood of water, once set up, has resulted very disastrously, and endangered the entire structure. In a shallow timber dock this danger is much less, but in those of great depth, which are necessary for the accommodation of large battleships and cruisers, it is a very formidable one and is ever present.

Leaks in a timber dry dock, particularly a deep one, are looked upon with grave suspicion and require constant watchfulness, because, once started, they are particularly dangerous in a dock of this kind, as its structure, not being watertight, silt and soil may be carried in, undermining portions of the structure, and loosening its hold upon the soil to such an extent as to cause collapse or fear of collapse. This was the experience in the case of Dry Dock No. 3 at the New York Navy Yard.

In a masonry dock the structure is designed to resist by its own weight the forces referred to, and, as its construction is watertight, it embodies all the elements of stability and safety. Any leak which may occur in a structure of this kind, due to a faulty joint, is not attended with the same danger as in the case of a timber dock. The early stone docks, now 65 years old, have had for a great many years some leaks through faulty joints, but these have existed without any material extension of the trouble, and without involving any instability or danger; and these docks have preserved their integrity and usefulness without failure or interruption of any kind.

It is of the utmost importance that a dry dock be absolutely safe. Its collapse might prove to be exceedingly disastrous to property and to life. A valuable war vessel, costing millions of dollars and containing hundreds of lives, should be placed in no dock which is not absolutely safe in every respect, and about which there is no doubt. The feeling of security in placing a vessel in a dry dock under such circumstances is worth the expenditure of any amount of money necessary to provide the very highest type. Accidents of more or less serious character have occurred in the timber docks, but none has ever yet developed in a masonry dock, and the advantages of the latter, in the matter of stability and safety, are immeasurably greater than in the case of a timber dock.

A dry dock is a great public work and one of great utility, and should be, like other works constructed by the United States Govern-

ment, of the best and most substantial quality, and an example, in every respect, of the best practice in constructions of its class. In its material the Navy is not content with anything which is not the equal of the same all the world over. All other first-class powers of the world are building graving docks of masonry of the best, most substantial and enduring type. Is it left for this Government to content itself with cheap, temporary structures, unstable, or embodying the elements of danger? Com. Endicott.

R. G. PACKARD, M. Am. Soc. C. E.—The comparisons made thus far have been between the timber dry docks of the present day and the granite dry docks of many years ago. In such locations as League Island and Beaufort, S. C., especially, the present concrete structures, made of high-class cements, would be much more satisfactory than the timber docks, or even the old masonry docks that have stood so well, the record of which has been given by Messrs. Cathcart and Endicott. It would be interesting to know the number of days the docks have been in working order, and the cost of repairs per foot of length, and per foot of draught. In other words, how much per ton does it cost the Government for dockage? The comparison of cost made by Mr. Endicott is based on the highest class of concrete, and with more or less granite lining of high cost, and highly finished. Such linings may be a necessary feature, but, in making massive structures, such as these concrete basins, a less costly class of materials could be used, more or less saving might be effected, and an equally good structure secured. Thus the relative costs of timber and concrete structures might be brought nearer together than they are to-day. Mr. Packard.

P. C. ASSERSON,* M. Am. Soc. C. E.—The speaker's experience in repairing stone docks and in repairing and building wooden docks has been quite extensive. His first experience in repairing stone docks was at Norfolk, Va., where the entrance work had been destroyed by the explosion of gunpowder. The cost of repairing that dock was not very great. The greatest expense was incurred in rebuilding the caisson, which had been destroyed by fire when the yard was evacuated by the Confederates, because, although the inner abutment was destroyed, the outer sill was not damaged, and the repairing of the stone abutment required no great engineering science. It took three years, but the appropriations were small. The speaker's next experience was in rebuilding the abutments and outer works of the stone dock at New York, which had been disintegrated by the water finding entrance, principally from the sea. The chief cause of the trouble was bad construction, something rather surprising. It was found when the abutments were taken apart and the masonry taken down to the sill level, that in the abutments proper there was not a single header. The stones were all stringers, the same as one Mr. Asserson.

* Civil Engineer, U. S. N.

Mr. Asserson. brick laid on another, without a heading course. That was probably one of the principal causes of its destruction. The second cause was the poor cement that had been used. Only ordinary Rosendale cement had gone into its construction. A third cause was that someone had attempted to remedy the leaks on top by caulking the joints with lead, thus forcing apart the stone and admitting more water. The speaker believed this to have been a fatal mistake, and thinks the same thing has also occurred in other cases where lead caulking has been used instead of cement. In the repair of the Brooklyn dock it was found also that the sides had settled and split the head of the dock in three parts. At the head of the dock cracks were found which extended from top to bottom on each corner, and in some places the speaker could run his arm into the crack. Of course the water found ingress there, and frost following it helped to displace the masonry. In repairing, these things were all taken into account, and care was taken to put in headers where the masonry had been loosened. The cement used was the best, not that this country afforded, but that could be imported from Germany. The speaker believes that there have been absolutely no other repairs upon that dock since these were completed in 1888. It is in perfect condition, and will probably remain so for many years.

In the construction of the wooden dock, No. 2, at Brooklyn, the speaker had some experience—what some engineers would call "a great deal of experience." The basin of the Wallabout is underlaid by what some call quicksand. The speaker calls it mica sand, but does not know that there is much difference. It is very quick, however, and there are "only 90 ft. of it." In places it was found that piles 50 ft. long could be driven very readily and very quickly, but if the hammer was taken off quickly, the pile would follow it up again. To stop the quicksand the contractor resorted to close piling, ballast stone, bags of concrete and all sorts of expedients. Finally, after a great deal of trouble, the concrete was put down, and, by the friction of the sand on the piles, sufficient stability was found in the bottom to hold any ship that the dock would take; but with the sides it was different. The speaker does not think it is generally known, but shortly after the dock was built it was found that in driving home the wedges at the ends of the shores when docking a ship, the dock would go back or spread on both sides as much as 4 or 5 ins. That happens even now whenever a vessel is shored. It is only a question as to how long the diagonal timbers will bear the continual thrust on them. The soil that should pack up to the back of the timbers, and make them steady, is of such a nature that it falls away from the back of the altars and diagonal timbers, leaving the sides of the dock without any support on the back except the piles. The speaker regards the dock as unsafe for docking a heavy ship that has any list.

As long as a ship is perfectly upright and balanced upon the keel blocks, all will be well. Mr. Asserson.

The speaker saw the construction of the timber dock at Newfoundland. The bottom consisted of shale rock, and as piles could not be driven successfully, a concrete bed, 6 ft. thick, was laid down, and formed a very good bottom.

About two years ago the speaker repaired the outer abutments, the sill and the sheathing on the abutments of the timber dry dock at Norfolk, Va., which had been destroyed by the teredo. The cause of this destruction was as follows: When the *Texas* was being built at Norfolk, she was put in the dock to reeeive her armor, and kept there some nine or ten months. Unfortunately the caisson had been put on the inner sill, thus leaving the outer sill and abutment exposed. During this nine or ten months the teredo completely riddled and demolished the outer side sills as well as the surface planking or timbering of the abutment proper. The sills were completely destroyed from high-water mark to within 1 ft. of the bottom. There, the sill had been covered by mud, and was, of course, in good condition. The covering of the wing walls, extending out on the side, and the end sill of the wing walls were entirely destroyed. These wing walls had been covered by creosoted planking underlaid with ship felting as a protection against the teredo, and that fortunate circumstance had saved the inner plank, but the outer plank was completely gone, and in places had commenced to fall off. In a year or two more the inner planking of the abutment and of the wing walls themselves would have been completely destroyed. By degrees the wing walls and the abutments were stripped, and, by local cofferdams, the whole of the front covering was renewed at a cost of about $10 000. A new sill on each side of the dock was put in and overlaid with a built-up Z-bar of steel, secured in such a way that the sill will probably not be disturbed by the teredo for many years.

The speaker does not know of any chemical preparation with which wood can be treated whereby it will be enabled to completely resist the teredo for more than 10 or 15 years.

The speaker is familiar with the structure of all the docks from New York southward. The timber used in the facing, and subject to decay, has been merchantable timber of good quality, and has been nearly all heart wood, where it was possible to obtain it. Of course, in structures of that kind, where large quantities of timber are used, there will be a little sap wood, and there is no doubt that such wood begins to decay at once. That was the case with the dock in Norfolk, where, two years ago, the speaker commenced to repair the upper works. The piling, the cross-cap timbers and the diagonal bracings were found badly decayed from the coping to the low-water line.

The spreading of the sides of the dock previously mentioned is due

Mr. Asserson. to the difficulty of backing the soil up against the back of the altars when building. Further, being a disturbed soil, on a slope of 35°, it settles away from the altars, and, of course, the space left fills with water. The same trouble does not occur with the stone dock, where the retaining wall is vertical on the back.

The wooden docks have had the same care as the stone docks. Those who have been interested in dock construction for the last 30 years have had great anxiety in respect to all the docks, and they have been taking care of them as fully as possible. It would be necessary to be more careful in docking a ship in a wooden dock than in a stone dock, and a ship can be handled in a much more rough and ready manner in the latter than in the former, without danger of injuring the dock.

In handling material in a wooden dock care must be taken not to destroy the altars, a necessity which does not obtain, to as great an extent, in a stone dock. Less care is required in wedging against a stone dock than against a wooden one, though the speaker does not believe that any wooden docks have been injured by docking a ship, so far as he knows, nor have any ships been injured on account of docking in a wooden dock. But as the docks decay, of course injury may happen.

The speaker thinks that from the nature of the two materials, stone and wood, equally good construction cannot be obtained, and it is hardly possible to make comparison as to strength or durability. Sufficient timber could not be put into a wooden dock to afford a proper comparison with a structure of stone and concrete. In the speaker's experience, the construction of some wooden docks has been rather on the slimsy order of mechanism and construction, though he thinks the Anglo-American Company worked honestly and did the best they could with their wooden docks. Undoubtedly, the builders of wooden docks did the best they knew how, and exercised care, except perhaps in the case of a certain timber dock recently constructed.

The speaker can furnish no data as to the relative cost of good concrete or good wooden docks. It is only when a contract is made with responsible people that the relative cost of the two classes of docks can be ascertained. Wooden docks can be built for $1 000 per lineal foot, and stone and concrete docks can be built for $1 500 per lineal foot.

In the early days the speaker was in favor of wooden docks. Now, he is in favor of stone, or, rather, concrete and stone. He believes that in the future dry docks should be constructed of concrete, the best that can be made, faced or lined with stone, particularly in the wearing parts of the dock. There are parts of the dock, the wings and the angles, for example, that need not necessarily be of stone, as concrete will answer the same purpose.

Mr. Asserson.

In the Norfolk dock the upper part was recently rebuilt entirely of concrete, which made a very good job. The only reason for putting a wooden lining on the front of the concrete altars was to prevent the abrasion of the green concrete. The upper part of the Norfolk dock was repaired while the dock was in daily use, and the speaker thought it was necessary to do something to protect the concrete during construction. If he had had the money, he would have put in stone instead of wood, but the latter was put in in such a manner that when the planks split or became decayed they could be taken off and replaced by others.

The speaker can see no reason why dry docks constructed wholly of concrete, and without stone lining, should not be satisfactory, if the corners of the altars are rounded off. He would prefer, however, to make assurance doubly sure, and use stone on the wearing parts, for the reason that there might be carelessness in shoring up ships. There is a probability that the corners of the concrete altars might be broken off, particularly where they are so shallow—only 8 ins. high—as in these structures. With a larger body of concrete, say 12 or 18 ins. high, the speaker would not hesitate to allow the use of altars wholly of concrete, because the corners of the altars themselves need not be touched. The corners of the concrete are sometimes broken off by sliding timbers and heavy material over them and down into the dock.

In reference to the lining of the timber docks with creosoted plank, it is a fact that the teredo will not attack it for several years. The teredo will not pass across a water-bearing seam. But creosote will protect only for a time. The speaker's experience has been that where creosoting has been throughly well done, a good, heavy, dead-oil creosote injected into the timber at a pressure of 100 or 120 lbs. per square inch, it will protect timber for from 10 to 15 years; after that the creosote is not so effectual. Unfortunately, the creosote used in the altars at the Norfolk dock was not the best kind. The speaker thinks it was what is called "pine oil," and he believes it helped to rot the timber more than to preserve it. However, the top of the dock at Norfolk is badly decayed for a distance of 8 to 9 ft. down from the coping.

The timber dry docks with which the speaker has had experience have all been constructed in about the same manner. There is a difference in the shape, but they have been practically earth excavations, having the sides lined with timber, the slope of the ground being such that it would stand by itself, except that at the Brooklyn yard the ground does not stand anywhere, but moves and sinks as soon as the mica sand has been reached.

Most of the Government's timber docks are good, but the speaker thinks there is room for improvement as to their strength, especially as regards the sides; the bottoms of the docks will no doubt last many years.

Mr. Asserson. There is no question in the speaker's mind but that in these days the Government should build for the future. Wooden docks were adopted by the naval service, not because of their durability, but because they could be built quickly and cheaply. Our need of docks was such that we could not wait for stone docks to be constructed. For 20 years Congress was utterly unwilling to appropriate large sums for dry docks, for repairs in the navy yards, or for building up the shops that were decaying.

Now that Congress and the country are willing to appropriate money, it is time to abandon that policy. Instead of building wooden docks, that serve at most only for 20 or 25 years, structures that will serve generations to come should be built.

The estimated cost of a concrete dock, 750 ft. long and lined with masonry, for Norfolk, Va., was $1 100 000. For a timber dock of the same size, and built as strong as possible, the estimate was $700 000. The estimate was for a concrete dock lined with stone 3 ft. thick, with stone abutments, and for a steel caisson, including pumping machinery. The stone lining was to have been set in the main dock chamber, far enough into the concrete to be firm and to form a tread or step, and with a very open slope—30 to 33°—so as to admit light and air, the same as for a wooden dock. There is no doubt that the shape of wooden docks is very good, because light and air are so readily admitted, and because access is so easy.

If first-class timber, or heart timber only, was put into a dock, and in a first-class manner, the dock would last 5 to 8 years longer than ordinarily, but all heart timber can hardly be obtained in such large quantities.

The wooden docks have served well to the present time, and the speaker thinks it is best to praise the bridge that carries us safely over. Nevertheless, he is of the opinion that we should now adopt material of a more durable character for structures that we expect to last and use for hundreds of years.

Mr. Tait. JOHN G. TAIT, Assoc. M. Am. Soc. C. E.—No one makes any attempt to deny the fact that a masonry dry dock is much more desirable as a government structure than a timber dry dock. Therefore the item of first cost must be the argument that influences the Congressional Committee on naval affairs when they decide to build timber instead of stone docks. As engineers, citizens and taxpayers, we are all interested in having the Government make the best investment, and build masonry docks, and to that end, until the Naval Committee are educated up to the advantages of stone docks, and while the engineers are still in competition with men advocating timber docks, any unnecessary cost should be omitted from the masonry docks.

At the site of the Boston dock the cost of concrete per cubic foot is less than half that of timber. This dock contains a great deal of gran-

ite, the stones are very deep, and, according to the plans and specifi- Mr. Tait.
cations the radial lines have to be preserved for the full depth of the joint, which makes very expensive cutting. Then, in addition to that, the same courses are continued from the main part of the dock out into the jaws, so that each stone constantly increases in width as well as being a voussoir.

These requirements probably make the cost of the dock about $200 000 more than it would have been if built entirely of concrete, or of concrete with granite quoins. If the dock had been designed with a smaller lining, and if there had been a different arrangement of the stones in the jaws, the bid would have been very close to that of a timber dock.

JOHN KENNEDY, M. Am. Soc. C. E.—Having recently examined all Mr. Kennedy.
the large dry docks on the Atlantic coast from Halifax to Norfolk, and on the St. Lawrence and Great Lakes, from Levis to Lorain, mainly with reference to the choice of material for the construction of a dry dock of large size at Montreal, the speaker will state some of the results of his observations.

The wooden dry docks of both public and private ownership show general weakness of construction, and a rather rapid decay. The decay is, of course, most rapid in the parts above water level, and it appears to reach farther down and be more rapid in docks in fresh water than in salt. For example, beginning at the North, the wooden dry dock at Portland, Me., built in 1869, has had no recent repairs, and its altars and pile heads above low water are entirely rotten.

At Boston the altars and other lining of the three wooden docks built in 1854 to 1864 have been practically all rebuilt inside, and have required frequent repairs. The largest one is leaking beneath the entrance. The two private docks at Erie Basin, Brooklyn, built in 1866, were all relined in 1881, and concrete substituted for wood in the upper five altars. The wooden dock at Cramp's, Philadelphia, built in 1876, has largely failed. The altars and the slopes near the abutments are bulged in badly at places; the altars, from weakness and decay, have been stove in at places, revealing cavities behind, from which the earth slopes have been washed down. The League Island dock of the United States Navy, built in 1891, is notably weak, and has bulged in badly. While occupied by the United States cruiser *Columbia*, a month ago, the abutments were shored up to prevent collapse. There are serious leaks of water and mud in the bottom and lower altars of one side, and the earth slopes behind the altars have run down in several places, leaving large cavities. Repairs, estimated to cost about $64 000, are about to be undertaken. The Norfolk Navy Yard dock, built in 1889, is now having the upper eleven altars renewed in concrete because of the decay, not only of the altars, but of the braces and pile heads which carry them. The

Mr. Kennedy. wooden dry docks visited on the Lakes are in much the same general condition.

The speaker found that all engineers of the United States Navy Department who have had experience in the working and maintenance of wooden dry docks, and with whom the question was discussed, were understood to be quite against wooden dry docks, at least for naval use, because of their weakness and short life. On the other hand, private owners of wooden docks are in favor of wood for their own service, and in adding a new dock they build either wholly or mainly of wood. The obvious difference of conditions will account for much of the difference of opinion. In Government docks a small saving in first cost is of minor importance, but weakness and frequent need of repairs are well nigh intolerable. On the other hand, in docks of private ownership, saving in first cost is usually of serious importance, while the cost of maintenance repairs is met by the earnings of the dock, and is less felt. It is to be noted, however, in the more recent private docks built by owners of experience, that there is a desire for something more permanent than wood. A notable instance is that of the large dock now being built by the Newport News Shipbuilding & Dry Dock Company, in which the abutments and wings are to be of concrete. In the Erie Basin docks, Brooklyn, the wooden altars above the water line have been replaced by concrete. It may be noted, too, that all the recent wooden docks, of which the speaker has knowledge, and the more ancient ones, also, have not been built under any patents, nor by contractors holding patents; or, in other words, while the building of wooden and partly wooden docks is still continued by private firms, the patented kind seems to have passed out of favor.

In Canada there are no wooden dry docks of importance. All the Government docks, three in number, are of stone and concrete. One of these, on the St. Lawrence, at Levis, Que., was finished in 1886; one on Lake Ontario, at Kingston, Ont., was opened in 1892, and the third, on the Pacific Coast, at Esquimalt, B. C., was finished in 1887. All are of excellent limestone ashlar, with concrete backing and inverts, and none have had any repairs of more importance than pointing, except in the caissons and other accessories.

Much reduction in the cost of stone dry docks has been made, by the substitution of concrete for part of the masonry, and it is proper to consider how much farther such substitution may be wisely carried, in cases where economy is of prime importance. The masonry of a stone and concrete dock is, of course, that in the linings, altars, copings, inverts, entrance and other parts where it is requisite that it be of cut ashlar, of expensive kind; and the further elimination of the whole or any considerable part of such costly work, rapidly reduces the average of the whole. There can be no doubt as to the strength

of solid concrete dock walls being fully as great as that of walls of concrete and ashlar combined, in which the bulk of the wall is of concrete and the ashlar is a mere facing. The real question is as to the wearing and weathering qualities of the inner surface and of the altars, if of concrete only. Examples of concrete of ample durability as well as of strength, can be pointed out, which show that good concrete will fulfill all needed requirements, even in northern climates. The making of Portland cement has also been so wonderfully improved, and the testing of it has become so simple and reliable, that there is now no doubt as to the obtaining of cement of an excellence and uniformity which was unattainable a few years ago. Mr. Kennedy.

In England and throughout Europe, dry docks, wet docks, arched bridges and other important structures, made almost wholly of concrete, are becoming common. At Erie Basin, Brooklyn, there is a minor, but, so far as it goes, a very satisfactory example of the durability of concrete, in the altars and copings of dry docks in this climate. The upper five altars of the two wooden docks were, as already stated, taken out and rebuilt wholly of concrete, and their condition now, after eighteen years of constant use, is excellent.

The Halifax, Nova Scotia, dry dock, of 580 ft. length, built in 1886 to 1889, is all of concrete, except the meeting faces of the caisson gate, the coping courses of the altars and main walls, and the stairways and material slides, which are of granite, and the linings of the culverts, which are of brick. The inverts of the entrance are all of concrete except the meeting faces. The example, though not wholly satisfactory as to the durability of the concrete, is a very instructive one. Considerable areas of the inner surface of the body walls have scaled off, and have been relined with blue bricks, but the failure is readily accounted for by much of the walls having been built under most unfavorable conditions as to storms and frost, and storage of cement. On the other hand, adjoining parts of the main walls, and all of the inverts and walls of the entrance, are standing excellently well. Altogether, the dock shows that really good concrete is a proper material for dock walls, even in a rigorous climate.

A notable example of a very large dry dock, with walls almost wholly of concrete, is the new No. 3 Graving Dock at Glasgow, designed by Mr. Deas, Engineer of the Clyde Trust. It is 880 ft. long, 115 ft. wide at top, 32 ft. depth from floor to coping, and is divided into two sections by gates. The side walls are entirely of concrete, except a thin backing of brickwork placed against the sheet piling which surrounds the walls. A peculiar and what appears to be an excellent method of construction of the walls is that their whole inner faces are made, not out of concrete built *in situ*, but of granolithic concrete blocks, separately molded and hardened, and then built into place as ashlar. The altars, up to about half the height of the walls,

Mr. Kennedy. are 3 ft. 6 ins. high and 20 ins. tread, and from that to the top of the walls are 18 ins. high by 14 ins. tread, the granolithic blocks being molded to the height of the altars in each case, so that a course of blocks builds an altar. The coping of the top of the walls is also of granolithic blocks. The head of the dock is semi-circular, without altars, and is lined with concrete blocks which are of heights corresponding to the adjoining altar courses, but these blocks, not having exposed tops, are made of ordinary concrete with only a granolithic facing 3 ins. in thickness. The cost of the dock and its accessories, all complete, including everything except the purchase of land, was £237 000 = $1 153 400, which is remarkably low for a double dock of such large dimensions.

An older example of a concrete dry dock in Britain is that at Newport, completed in 1890, of 350 ft. length by 30½ ft. depth, and, although built on a very difficult site, its cost was only $121 500.

An important example of concrete construction, for purposes analogous to those of dry docks and exposed to somewhat severe weather conditions, is that of the locks of the Trent Canal in Ontario, now being built by the Government of Canada.

The advantages in strength, durability and economy of maintenance of dry docks of stone, and of stone and concrete, over those of wood, have been ably and fully shown in the discussions presented; and, for the reasons just outlined, the speaker adds a plea for dry docks built almost wholly of concrete, as possessing much of the advantages of both.

Mr. Ritchie. JAMES RITCHIE, M. Am. Soc. C. E.—The speaker built a dry dock recently, but it was of wood, as is customary on the lakes. He suggested that the abutments be built of stone, but the cost of such work was the objection made by the owners, and only a small amount of concrete was used. It is a large dock, 560 ft. long and 100 ft. wide, but, as has been stated, it has the one advantage of a wooden dry dock—it is shallow, only 17 ft. on the sills; consequently it is considered perfectly safe, and has so far been extremely satisfactory. For a dock of great depth, or for docks where the first cost is not the essential matter, stone, or concrete lined with stone, is, in the speaker's opinion, the only proper material to use.

Mr. Collingwood. FRANCIS COLLINGWOOD, M. Am. Soc. C. E. (by letter).—The original cost of wood construction is, of course, less than stone. The repairs and renewals are a matter of experience, and it is claimed by those having long experience with wooden docks, that the interest on the difference in cost between them and stone will more than pay for keeping the former in first-class condition. The reduced rate of interest now prevailing, however, may cause a change in this regard.

As to convenience, etc., those who are constantly using docks are most competent to speak. Such matters as length of shores and con-

venience in adjusting them, convenience in handling material and, in movements of men, slipperiness and cold to the feet, contents to be pumped out, and other practical matters have to be considered. As the writer's connection has been with construction only, others can speak with more authority. Mr. Collingwood.

It is to be regretted that the experience in building some of the recent timber dry docks has in a measure thrown discredit upon them. Errors in judgment in engineering matters are not justly chargeable against a system of construction which has certainly proved successful in many existing examples.

CHARLES H. HASWELL, M. Am. Soc. C. E. (by letter).—The writer thinks it is to be regretted that the subject under discussion is confined to the relative fitness of the material of a dry dock, inasmuch as piers and bulkheads, erroneously termed docks, are of more general requirements and consequent utility than docks either wet or dry. Custom and some lexicographers term the enclosed space between two piers a dock, whereas it is strictly a basin, although colloquially, whether the structure is a single pier or a bulkhead, it is termed a dock. Mr. Haswell.

If the subject had included piers and bulkheads, whether of masonry, crib-work or rip-rap, the writer would avail himself of the opportunity to refer to their design and construction, on the basis of his experience and observation; but as the subject is confined to the question of the relative merits of stone and wood for the construction of dry docks, it is a restricted one, as it embraces only the relative cost and endurance of such structures.

If the period of construction and the first cost are controlling elements, wood alone can be entertained, as it involves less cost of material and labor in construction, not only in its walls, but in its foundation. Its defective elements are decay and damage by fire, involving the probable damage or destruction of a vessel under repairs.

If, on the other hand, the period of its construction and its first cost are not restricting elements, and a permanent structure is required, stone or concrete is not only indispensable, but, in the matter of ultimate cost, it is questionable if frequent repairs, and the periodic renewal of a wooden structure, would not involve a greater cost than stone.

FRANCIS T. BOWLES, Esq.* (by letter).—After thirteen consecutive years' experience in the active use of wooden and stone dry docks at the Navy Yards at Norfolk, Va., and New York, and after having had sole charge of the repairs to the wooden dry dock, known as No. 3, at the latter place, in which it was necessary to reconstruct the whole entrance, the writer wishes to express an unqualified preference for a Mr. Bowles.

* Naval Constructor, U. S. N.

Mr. Bowles. masonry or concrete dry dock over wooden construction. In testifying before a Congressional Committee about a year ago, the writer expressed the following opinion:

"I believe that a wooden dry dock bears the same relation to a stone dry dock or concrete dock, that a wooden trestle or a wooden bridge bears to a steel bridge or stone culvert. I consider it a temporary structure."

This opinion appears to present the case fairly.

The wooden dry docks built 10 or 12 years ago for the Navy have served their purpose well; they were cheap, and admitted of easy and rapid construction by unlearned persons. They were built at a time when little attention was given to the subject of dock construction in this country, and when materials for stone and concrete work were very expensive. Moreover, these docks supplied an immediate and pressing need which at the time could not have been otherwise met. It is now known that the deterioration which they have developed and the essentially temporary character of the work and material of their construction, show that their continued use involves danger and great expense for repairs.

Consideration of the most elementary principles of engineering leads to preference for permanent rather than temporary construction where cost is not prohibitive, and at the present time the cost of concrete and stone construction is sufficiently low to require of the Government the most durable, safe and best construction of its dry docks.

This opinion is not derived from the recent bids on the new dock at Boston, which, to the writer, appear to be inconclusive. He is strongly opposed to the letting of contracts for the building of a whole dry dock to one person or corporation, on the ground that the nature of the undertaking involves too many undetermined elements to enable any person to make a bid which will enable him to do the best work throughout. The work should be under the sole charge of a competent engineer, who should control absolutely the conduct of the work, and let contracts from time to time for definite portions, on which clear and definite specifications can be made. That is good business, and in the end will produce the best results. This digression from the main subject will possibly be pardoned, because, in advocating stone or concrete docks, it is the writer's opinion that the engineering problems are great, and must be treated by modern methods.

Mr. Boller. ALFRED P. BOLLER, M. Am. Soc. C. E. (by letter).—It is astonishing that a controversy regarding the relative merits of stone and wooden dry docks should be given serious consideration by a great government like that of the United States. That such a controversy does exist is a sad commentary on the intelligence of the source of ultimate power, the National Congress. There are some questions that have only one side, and this is one of them. It is so thoroughly one-sided that only the crass selfishness of interested parties, operating in some mysteri-

ous way upon Congress, forces a consideration of the subject at all. Is there another civilized government under the sun that would thrust aside the united demands, yes, pleadings, of its technical experts to do the right thing, simply for the sake of maintaining a special private interest, whose business is building wooden docks? Mr. Boller.

Stone or wooden docks! As well ask our railways to build wood instead of stone or steel bridges. The only so-called arguments advanced for wooden dry docks are those of economy and expeditious construction. Economy of first cost is one thing, and economy of endurance and maintenance is another. Rapidity of construction is a matter of little importance for a Government work, compared to solidity and endurance, but modern methods of executing work permit of such rapidity of performance that almost any reasonable expectation of time can be met. What if it does take twice as much time to build a stone dock as a wooden one; when the work is done it is done for all time practically, and it can be operated with absolute confidence and freedom from the anxiety that always attends a wooden dock. Absolute safety and economy of endurance are the cardinal requirements for any dry dock suitable for the purposes of a modern navy, with its deep-draught battleships, which can only be met by using imperishable material. To build such docks of wood, that commence to depreciate from the day of their completion, deteriorating increasingly with the years, until absolutely worthless, is serious folly that is belittling to our naval administration, a waste of the public funds, and a source of ridicule with competitive nations. A wooden dry dock is all well enough for temporary purposes, or to meet the necessities of an impoverished government, or the financial limitations of a private party. The United States is in no such impecunious position, and is expanding into a naval power, the limits of which no man can foresee. This country is destined to a restoration of a merchant marine that will recreate the long-lost glory of the seas, now only a reminiscence, and on a scale commensurate with the activities of a hundred million of people, backed with unlimited resources. For the Government of such a nation to even talk of building wooden dry docks, is preposterous. While it is gratifying to know that the comparative cost between the two systems of docks shows, perhaps, 25% difference (according to recent tenders), the question of first cost, to the writer's mind, cuts no figure at all. The United States cannot afford to build wooden docks, even on a simple financial comparison, as the statistics so elaborately discussed by the naval speakers, demonstrate beyond cavil. Whether stone dry docks cost much or little, whether they take twice as long to build as wooden ones or not, there is but one thing for the United States Government to do, and that is to build stone docks, even at the risk of hurting the feelings of existing wooden dry dock companies.

Mr. Mohun. EDWARD MOHUN, M. Am. Soc. C. E. (by letter).—The writer is thoroughly in accord with those who hold, that while a commercial company, or private individual, may be justified, from financial considerations, in the use of timber for these works, yet that a great and wealthy maritime nation would be ill advised if, through the parsimony of its rulers, other than the best and most enduring material were made use of in constructions on which in time of war the efficiency and possibly the very existence of its fleet might depend.

The dry dock at Esquimalt, B. C., was opened in July, 1887; H. M. S. *Cormorant* being the first to enter.

Its dimensions are: Length of floor, 450 ft., which can be increased to 481 ft., by berthing the caisson at the outer invert; breadth of entrance, 65 ft.; breadth at coping, 90 ft.; breadth of floor, 41 ft.; depth on sill at ordinary high water, 26½ ft., and at high spring tides from 29 to 30 ft.

The body of the dock is constructed of concrete faced with masonry.

From the bills of quantities there were used in its construction: 30 639 cu. yds. of concrete, 6 556 cu. yds. of brickwork, and 109 887 cu. ft. of masonry.

Its cost was about $900 000.

It has been in almost constant use for nearly 12 years, and the writer is informed by the Dock Master that the repairs to the body of the dock, during that period, have been effected at the cost of five barrels of cement and a week's wages to a mason.

During the last years the cost of replacing defective work on the caisson chamber and pump wells has been about $4 000. Owing to the records having been destroyed by fire, it is impossible to give the correct figures. Much of this expenditure appears to have been caused by the use of brickwork in the pump wells and entrance works. The writer regrets his inability at the present moment to furnish the actual cost of these repairs, but it appears reasonable to assume that, had the same material been used as in the body of the dock, their cost would have been reduced to a minimum.

The pumps have been furnished with two new sets of rubber valves, and a new pinion and spur wheel have been added to the machinery.

Irrespective of the tendency, of late years, to increase the size of warships, it would seem, that to fulfill all the conditions under which its services might be needed, a dry dock should be long enough to take the longest vessel in the fleet, wide enough at the entrance to take that of the greatest beam, and with depth on the sill to take that of the greatest draught.

The following are the dimensions of the largest ships in the British Navy:

Battle-ships, *Majestic* class: Length, 390 ft.; beam, 75 ft.; draught, 27½ ft. There are nine of this class. Mr. Mohun.

The *Powerful* and the *Terrible* are the two largest cruisers of the British Navy afloat, and have a length of 500 ft., a beam of 71 ft., and a draught of 27 ft.

It appears to the writer that the least dimensions of dry docks for the use of the British Navy at outlying stations would be: Length of floor, 500 ft.; width at entrance, 80 ft., and depth on sill, 30 ft. Probably few engineers would recommend the construction of timber docks of such dimensions to receive battle-ships of 15 000 tons displacement.

A few weeks ago the writer was discussing with the captain of the flagship on this station the size of docks required by the later additions to the British Navy, and that gentleman pointed out that under certain possible conditions a very great depth on the sill might be needed. It must be understood that this was simply in conversation and possibly might not be the result of mature consideration on his part.

For instance, suppose a vessel with her bows stove in and her forward compartments full of water, the result perhaps of a naval action or collision; it is obvious that she would be down by the head possibly 10 ft. more than her normal draught, and to enter the dock would need a great depth on the sill. The officer above referred to suggested that a depth of 40 ft. on the sill might be required in such an emergency.

To the writer this is an entirely new view, on which he does not feel competent to express an opinion.

A case somewhat in point, however, occurred here a few years ago. The second-class cruiser, *Amphion*, 4 300 tons, with a usual draught of 22½ ft., was taken into dock after ripping up her bottom on a rock. She reached the dock at high water, and was just able to enter, grazing the sill.

The Admiralty grant toward the construction of the Esquimalt dock was $250 000, and this single use of the dock probably recouped the amount and possibly saved a vessel to the British Navy.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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ADDRESS AT THE ANNUAL CONVENTION AT CAPE MAY, N. J., JUNE 27TH, 1899.

By DESMOND FITZGERALD, President, Am. Soc. C. E.

The first Annual Convention of this Society, held in June, 1869, was the beginning of a long series of conventions, held uninterruptedly from that time to the present, which have proved to be of the greatest benefit, not only to the Society, but to the profession of the civil engineer. At about that time applications for membership were falling off, and there were other premonitory symptoms of weakness. The Convention was designed to bring members together and to strengthen the chain which was to bind the profession into a firmer union. The designer builded better than he knew, for this great annual meeting has ever since acted as one of the most powerful influences in our progress.

The Annual Convention was also planned for the reading and discussion of professional papers, for the improvement of the members and for the encouragement of social intercourse. It does not appear under which of these heads the Address of the President may be included, but whatever its effects upon others or upon the profession, there is little doubt of its effect upon the occupant of the office. It seems highly probable that one filling this exalted station might not only survive all the trials that beset the way, but might even go so far

as to enjoy the distinction, were it not for this consumer of midnight oil—the Annual Address. For the present incumbent, the choice of a subject became unusually difficult from the fact that his immediate predecessors in office had so ably covered the field. In 1895, in Boston, the meaning and position of the profession and the Society, and the new epoch of the manufacture of power, were discussed by Mr. George S. Morison with a power of analysis which is rarely equalled; in 1896, at San Francisco, Mr. Thomas C. Clarke entered the lists with his exhaustive relations between science and engineering; in 1897, at Quebec, Mr. Benjamin M. Harrod delivered one of the ablest of our long series of addresses, on the relation of civil engineering to the progress of the present time and country, of its opportunities and limitations, and the responsibilities and duties of our Society; and last year, at Detroit, Mr. Alphonse Fteley drew from his rich experience for a review of the progress of engineering. After such intellectual feasts it may not be wondered at if one in this position asks himself : "Are there any crumbs left for me to distribute ?"

Those who have had occasion to study the early condition of engineering in this country and the lives of the fathers of the profession, must have been struck with the paucity of the literature of the subject. As we are apt, in the feverish hurry that surrounds the engrossing pursuits of the present day, to overlook the debt we owe to the founders of civil engineering in the United States, and as the close of the century is an appropriate time for historical review, I have concluded to call your attention to a few of the earliest public works constructed in this country, and particularly to the men who designed and built them.

The history of engineering in America is naturally divided into four periods:

First.—That from 1785 to 1800, embraced in the earliest public works of the country of the nature usually undertaken by the civil engineer.

Second.—That from 1810 to 1830, beginning with the revival of works of internal improvement after a long period of inaction, and extending to the invention of the railroad.

Third.—That from 1830 to 1848, beginning with railroads and extending to the formation of the engineering societies.

Fourth.—That from 1848 to the present time.

Only a rapid survey of the first three of these periods will be attempted.

Before the Revolutionary War of 1776, the spirit of internal improvement had already begun to make itself manifest throughout the Colonies in various directions. Nowhere had this spirit taken more definite form than on the banks of the Potomac.

To John Bellendine, of Fairfax County, Virginia, a person described as possessing skill and judgment in water-works, is probably due the credit of first putting in definite form a scheme for connecting the Potomac River with the Monongahela, and the James with the Kanawha. Bellendine came as near being a civil engineer as any one else in the country at that time. He published, in 1773, his plans for constructing canals and locks for the improvement of the navigation of these rivers. Washington, who was almost an engineer, had already planned projects for connecting the Atlantic Coast with the almost unknown wilds beyond the Alleghenies, but war stopped the hand of progress in public works, and it was not until peace had been declared that the country once more turned to works of internal improvement. Here Washington again took a leading part, and by his exertions was largely instrumental in procuring from the legislatures of Virginia and Maryland a charter incorporating the Potomac Company, formed for the purpose of opening the waters of Chesapeake Bay to navigation from the interior rivers of the States, and ultimately to connect with the Ohio River. This was the earliest charter for any work of considerable magnitude worthy of being classified under engineering. Washington was elected the first president of the company in 1785, and, after subscriptions of stock, the work of improvement began. For some time this was confined to the removal of rocks from the bed of the Potomac River as far as the Great Falls, but the real difficulties of surmounting the 76 ft. of rise in these falls called for the services of an engineer.

In a letter to Edmund Randolph, dated September 16th, 1785, Washington wrote:

"And as the Great Falls are tremendous and the navigation thereof, in whatever manner it is attempted, will require much skill and practical knowledge in the execution; we propose, before this is undertaken, to invite a proper person from Europe, who has been employed in works of this kind, as a superintendent of it."

It is probable that some foreign engineers advised and perhaps designed the locks around the Great Falls.

Work on the Potomac Canal languished toward the last of the century, and, in common with all the other great undertakings of the day, slept until awakened many years later, when the Chesapeake and Ohio River Canal, which absorbed all the works of the old Potomac Company, was formed in 1823.

On January 5th, 1786, an Act for cleaning and improving James River was obtained. Under this Act, the old James River Company constructed a canal around the falls on James River, extending from Richmond to Westham, a distance of about seven miles, and improved the bed of the river by sluices as far as Buchanan.

The Dismal Swamp Canal, extending from a point on the Elizabeth River, near Norfolk, to Albemarle Sound in North Carolina, and passing through the Great Dismal Swamp, in the States of Virginia and North Carolina, was chartered in 1787, and a large amount of work was done before the close of the century; but progress was so slow that the actual connection of the waters of Chesapeake Bay with Albemarle Sound was not made until after the War of 1812.

The operations of the James River Company and the Dismal Swamp Canal, undertaken at about the same time as the Potomac Canal, are so similar in many of their features that they need not be described. The larger part of these enterprises was in the nature of river improvement, combined with lockage at the rapids or falls.

In 1784, South Carolina started a project for the construction of the Santee Canal, the first true canal in the country, of which the name is now almost entirely forgotten. This canal was 22 miles in length, and was built for the purpose of connecting the Santee River with the tidal headwaters of Cooper River, which empties into Charleston Harbor.

The charter for the Santee Canal was obtained March 22d, 1786. Work was begun in 1793, and the canal was finished in 1800. As this was the first work of internal improvement constructed in this country, antedating the famous Middlesex Canal by about three years, it possesses an unusual interest in a consideration of the history of engineering in this country. The Chief Engineer and Director was Colonel Christian Senf, a Swedish engineer, who was at that time Chief Engineer of the State of South Carolina. The total rise and fall

of the canal was 103 ft., overcome by eight single and two double or coupled lift-locks, varying from 5 to 10 ft. lift, and built of stone or brick masonry with wooden gates. The boats were 54 ft. long, 9 ft. beam and 2½ ft. draft. The average dimensions of the canal were 35 ft. wide at top, 20 ft. at bottom, and 4 ft. depth of water. The canal was furnished with 25 flood gates, several overfalls, abutments and walls for bridges, and for 13 miles of its course the digging was 15 ft. deep. The work was constructed by negro laborers, two-thirds of them being men and one-third women, and they were paid £15 to £16 per year.

The reports of the engineer are exceedingly interesting. The following is an extract from one of them:

"You and the Board of Directors will be pleased to remember that from the beginning of the work I have never been able to meet with a tradesman or any other person to assist me who had ever been employed upon similar work. Of course, being obliged to lay out every kind of work myself, and to direct its execution, I have not been able to give you from time to time my progress, as is usual."

There is something heroic in the simple descriptions of this engineer who alone undertook to carry his work to completion in the face of fevers, lack of facilities, and every conceivable obstacle. There is nowhere a reference to its being the first work of its kind in the country, no bombast, no flourish of trumpets, only the straightforward reports of the honest engineer striving to bring order out of chaos. The summit level of the canal was 5 miles in length and was to have been fed by reservoirs and swamps, which, however, afterward proved insufficient, and this was one of the principal causes of the failure of the canal.

In 1880, Colonel Q. A. Gillmore was requested to report upon the re-opening of the canal, under authority of a provision in the Land and Harbor Bill. The report was not favorable, but the engineers found the canal almost intact, although covered with a growth of timber and brush. The locks were generally in place, with the exception of the gates, which had gone to decay. On the advent of railroads, the canal was abandoned. The original cost of the canal was $650 667.14, and the ultimate cost $1 100 000.00. The company owned 110 slaves and 6 000 acres of land.

A project which was proposed and discussed long before the Revolutionary War, in fact by Penn, in 1690, was that of uniting

the Schuylkill and Susquehanna rivers by a canal. The first explorations were made by a Committee of the American Philosophical Society in 1770. The same route was surveyed again, a little later, by the astronomer, David Rittenhouse. In 1789 a few public spirited citizens formed an association and proposed various plans to the Legislature, and in the following year commissioners were appointed to explore several routes. This action resulted in the formation of two companies.

The first was the Schuylkill and Susquehanna Navigation Company, incorporated in 1791, to open a communication between the Schuylkill and Susquehanna rivers from Reading on the Schuylkill to Middletown on the Susquehanna.

The second was the Delaware and Schuylkill Navigation Company, incorporated in 1792, to open a canal from Philadelphia along the east branch of the Schuylkill, 16 miles, to Norristown.

As soon as the companies were organized, committees were appointed to superintend the commencement of the respective works. Unfortunately there was no engineer at that time in Pennsylvania competent to aid the committees; they, therefore, sent to England and engaged Mr. William Weston, an engineer of reputation, to come out and manage both works. In the meantime, the committees did not think it proper to remain idle. They commenced the respective undertakings, and had each well under way when Mr. Weston arrived, in January, 1793.

Early in the following year Mr. Weston made a report on the Schuylkill and Susquehanna navigation, to the effect that the unerring test of experience had proven the inefficiency of undertaking to open water communication by the beds of the creeks, and recommended the construction of a canal 70 miles in length from the Schuylkill to the Susquehanna. The expense he estimated at $821 330.60, in addition to the $400 000 already subscribed.

The company soon became convinced that Mr. Weston was right. Anticipating legislative aid, they worked under Weston's guidance through 1794. In December, 1794, five locks of 6 ft. fall and two road bridges had been completed, and progress had been made on the sixth lock. In this state work was suspended, with $250 000 spent.

In December, 1794, Mr. Weston made a report in regard to the Delaware and Schuylkill project, in which he said: "That one-third

of the work, about 5 miles, was finished, and that contracts were formed and forming for a vigorous prosecution of the remainder," and "that the whole canal of 16 miles could be completed for the sum authorized, $400 000," and, "if the stockholders would punctually pay the sums subscribed by them, a toll for a part of the canal might be drawn in 1796, and the whole completed in 1797 or 1798." In spite of every effort, however, the work languished, and in 1796, when nearly 6 miles of the canal were navigable, the work entirely ceased. The sum of $221 710.06 had been spent on this undertaking.

In 1811 these two companies were galvanized into life under the name of "The Union Canal," but no active operations were begun until 1821, when this historic work was begun and pushed to completion, in 1827, sixty-five years after the first survey, at a cost of $1 600 000.

The Schuylkill navigation was among the earliest works of this period. It was chartered in 1815, and work was commenced in 1816. The entire length of works controlled by the company, including slack-water navigation, was 108 miles. There were 31 dams and 120 locks in this distance, and the total lockage was 620 ft. Navigation was opened in 1825 for boats carrying 18 tons. The engineer was Thomas Oakes, and some of the works built by him are still in use. Oakes must have been an engineer of ability, and he probably came from England.

George Duncan, a Scotch engineer, who succeeded him, built the old tunnel below Pottsville and also the Tumbling Run reservoir system of the Schuylkill navigation, consisting of earth dams, one 48 ft. and the other 60 ft. in height. Duncan also designed and built the dams in the river above Reading, and the Duncan Canal, all now in use and described as admirable works.

While enthusiasm for the constructien of work of inland navigation was aroused further to the South, the great State of New York was not idle. Many years before the close of the eighteenth century public attention had been directed toward the great valleys of the Mohawk and Hudson as furnishing a means of water communication between the City of New York and the Great Lakes. The first steps were not taken, however, until March, 1791, when an Act was obtained to open communication between Wood Creek and the Mohawk River, and also providing for a connection between Lake Champlain and the

Hudson. The latter was known as the Northern Canal and the former as the Western Canal. Under this Act, Mr. Abraham Hardenbergh, a skilful and experienced surveyor, assisted by Mr. Benjamin Wright, made the survey in September and October, 1791. Mr. Wright was destined later to a career of fame as one of the first native civil engineers in America.

In March, 1792, an important Act was passed by the Legislature of New York providing for "opening a lock navigation from the navigable part of Hudson River, to be extended to the Seneca and Ontario Lakes." This legislation gave practical form to the idea of connecting the Hudson with the Great Lakes.

The Western Inland Lock Navigation Company immediately began active work on the improvement of the Oneida Lake, Mohawk and Seneca Rivers. These waters were opened from Schenectady to the Seneca Falls in 1796 for navigation by boats of 15 tons burden. The first locks were built of wood, as were those on the Potomac, and their decay and failure proved a serious blow to the stockholders.

William Weston was employed as engineer, and he rebuilt the locks with masonry, but at such a cost as to exhaust the resources of the company. It is stated that the wooden locks at the Little Falls, German Flats and Rome went to decay in about six years, and that they were rebuilt with brick, which again failed owing to the quality of the mortar, which was probably of lime.

Following closely on the construction of the southern canals came the great undertaking of that day in New England—the Middlesex Canal, which had the honor of being the second real canal constructed in the United States.

The charter was obtained in June, 1793, under the title of "Proprietors of the Middlesex Canal." Samuel Thompson, of Woburn, afterward Count Rumford, was engaged to run the levels, but the accuracy of his work was doubted by the directors, and on March 1st, 1794, Col. Loammi Baldwin was authorized to proceed to Philadelphia and secure the services of Mr. Weston, who ran the levels in July, and on August 2d endorsed the feasibility of the canal and estimated the expense at $500 000. It was decided to proceed with its construction, and Col. Baldwin, the father of three distinguished civil engineers and the discoverer of the Baldwin apple, was appointed Superintendent.

The Merrimac River flows in a southeasterly direction from New

Hampshire, and, when within about 27 miles of Boston, abruptly turns to the northeast. It was intended, by means of the Middlesex Canal, to unite this angle of the river with Boston, and provide a market for the commerce of the river.

The beginning of the canal was at a point on the Merrimac River about 2 miles above the present City of Lowell, which at that time did not exist. Its length, to the Charlestown mill-pond, the easterly terminus, was 27¼ miles. It was 30 ft. wide and 4 ft. deep; on its course were 7 aqueducts, 20 locks and 50 bridges. The Concord River was 37 ft. higher than the Merrimac and 107 ft. above tide, and was ample in volume to feed the canal in both directions. A stone guard lock was located on each bank of the Concord, with a floating tow-path between. In the Merrimac itself there were three stone locks. Many of the structures were built of timber supported on stone, and these structures, which soon began to decay, were the source of much embarrassment and expense. The canal, to-day, can be traced, but not without difficulty. Portions of it have been absorbed into highways; sewers and water pipes are laid in its bed; and, at Wilmington, the stone sides of a lock form the cellar of a house.

The water was let into the canal in 1802, and it was opened for public use in 1803.

The managers had included in their far-reaching plans the completion of the navigation of the river for 80 miles, and actually built many other smaller canals around the falls and rapids of the Merrimac. These works required the constant expenditure of money. Some idea may be had of the far-reaching views of the projectors of the Middlesex Canal when it is considered that they looked forward to a direct connection by water with Concord, N. H., and thence, by way of Sunnapee Lake, to the Connecticut at Windsor, Vt., and thence to the St. Lawrence and Canada. Their dreams, however, were destined never to be realized. The building of the Boston and Lowell Railroad parallel to the canal was the signal for its decline; it was closed in 1846 and its charter forfeited in 1859. The total cost of the canal was $613 000. The price of labor was $10 a month and board.

Inns were built along the route, where the boatmen congregated at night, as the canal was not allowed to be used after dark. Black strap, a mixture of rum and molasses, was the favorite drink of the boatmen and was sold at the price of 3 cents per glass.

It is related that Daniel Webster was engaged by the directors to plead one of their land damage cases of unusual interest, and that they paid him $100 as a retainer. As the day of trial drew near an effort was made to reach Webster, but he was always away, either in New York or Philadelphia, and the manager who told the story plaintively added: "Our one hundred dollars has gone to profit and loss account."

Some idea of the discomforts attendant on river and canal travel in the early days may be formed from the following extracts from contemporaneous sources. The first is from an advertisement, printed in Cincinnati, of the first regular packet line between Pittsburg and Cincinnati, in 1794:

"Two boats for the present will start from Cincinnati to Pittsburgh and return to Cincinnati in the following manner, viz.: First boat will leave Cincinnati this morning at 8 o'clock and return to Cincinnati so as to be ready to sail again in four weeks from this date. Second boat will leave Cincinnati on Saturday, the 30th instant, and return as above; and so regularly, each boat performing the voyage to and from Cincinnati to Pittsburg once in every four weeks. The proprietor of these boats having maturely considered the many inconveniences and dangers incident to the common method hitherto adopted of navigating the Ohio, and being influenced by a love of philanthropy and a desire of being serviceable to the public, has taken great pains to render the accommodations on board the boats as agreeable and convenient as they could possibly be made. No danger need be apprehended from the enemy, as every person on board will be under cover made proof to rifle balls, and convenient port-holes for firing out. Each of the boats is armed with six pieces, carrying a pound ball; also, a good number of muskets, and amply supplied with ammunition, strongly manned with choice men, and the master of approved knowledge."

The second extract is taken from a "Sketch of the Civil Engineering of North America," by David Stevenson, 1838:

"The canal travelling in many parts of America is conducted with so little regard to the comfort of passengers as to render it a very objectionable conveyance. The Americans place themselves entirely in the power and at the command of the captains of the canal-boats, who often use little discretion or civility in giving their orders, and strangers who are unaccustomed to such usage, and would willingly rebel against their tryanny, are in such cases compelled to be guided by the majority of voices, and quietly to submit to all that takes place, however disagreeable it may be. About eight o'clock in the

evening every one is turned out of the cabin by the captain and his crew, who are occupied for some time after the cabin is cleared in suspending two rows of cots or hammocks from the ceiling, arranged in three tiers one above another. At nine the whole company is ordered below, when the captain calls the names of passengers from the way-bill, and at the same time assigns to each his bed, which must immediately be taken possession of by its rightful owner on pain of his being obliged to occupy a place on the floor, should the number of passengers exceed the number of beds, a circumstance of very common occurrence in that locomotive land. I have spent several successive nights in this way, in a cabin only 40 ft. long by 11 ft. broad, with no less than forty passengers; while the deafening chorus produced by the croaking of the numberless bull-frogs that frequent the American swamps was so great as to render it often difficult to make one's self heard in conversation, and, of course, nearly impossible to sleep. The distribution of the beds appears to be generally regulated by the size of the passengers; those that are heaviest being placed in the berths next to the floor. The object of this arrangement is partly to ballast the boat properly, and partly, in the event of a breakdown, to render the consequence less disagreeable and dangerous to the unhappy beings in the lower pens. At five o'clock in the morning, all hands are turned out in the same abrupt and discourteous style, and forced to remain on deck in the cold morning air while the hammocks are removed and breakfast in preparation. This interval is occupied in the duties of the toilette, which is not the least amusing part of the arrangement. A tin vessel is placed at the stern of the boat, which every one washes and fills for his own use from the water of the canal, with a gigantic spoon formed of the same metal; a towel, a brush and a comb, intended for the general service, hang at the cabin door, the use of which, however, is fortunately quite optional. The breakfast is served between six and seven o'clock, dinner at eleven and tea at five."

The foregoing descriptions embrace the principal public works connected with the early history of the country. As a rule, they were unsuccessful, financially. Under the high-sounding title of "Lock and Canal Navigation," they consisted generally of efforts to improve the navigation of rivers by a removal of boulders, bars and other obstacles. Training walls and chutes were also built to concentrate the flow of the water, and often the first freshet removed all traces of the work which had been prosecuted at a considerable cost in time and money.

The primitive idea was to turn the beds of the streams into canals, and to overcome the rapids and falls by locks or inclined planes. The

latter certainly required the services of the trained engineer, and when the settlers could no longer prosecute the work without such service, recourse was had to Mr. Weston, from England, or to other foreign engineers. With financial losses, it is natural that discouragement should follow, and that several years should elapse before the struggle was renewed.

The great revival of works of internal improvement began in 1810, just before the beginning of hostilities between the United States and Great Britain, and this revival really marks the birth of engineering as a profession in this country. Up to that time, as we have seen, public works were undertaken in a spasmodic manner and without definite design. Ideas were abundant, and many of them were wonderfully far-reaching, but the means to carry them into effect, and the men to clothe them with practical form were both lacking.

The great work which was to create the new profession of the engineer was the Erie Canal. In one sense this is true, in another, the profession created the Erie Canal, for the spirit of the engineer was in the air, and would have made itself manifest in some other form. It so happened that the Erie Canal, a magnificent undertaking for its day, had the honor of being the first great pioneer work of the American engineer. We must, therefore, take a brief glance at the work itself. It will not be possible to follow the history of the canal, or to examine with any minuteness the many interesting events connected with its execution. Fortunately, they have been faithfully recorded, and can be found without difficulty. Whether one man or another is entitled to the glory of first pointing out the possibility of a connection between the waters of the Great Lakes and those of the Hudson does not enter into this discussion. We are more concerned in knowing how such a great project, with a multitude of details, all requiring the skill of the engineer, could have been carried to a successful termination in a land where there was apparently no engineer capable of the task.

Some of the preliminary surveys were made as early as 1808, but it was not until July 4th, 1817, that the actual work of construction was begun. It was finished in 1825 and opened with great pomp and parade. The total length from Albany to Buffalo was 363 miles. The canal was 40 ft. wide at the top, 28 ft. wide at the bottom, and was 4 ft. deep. There were 84 locks on the main line, each 15 ft. by 90 ft.

The total lockage was 688 ft. On the canal were 18 aqueducts, some of them fine stone structures, notably that over the Genesee River which was 804 ft. long and contained 11 arches. Among the aqueducts across the Mohawk there was one with a length of 1 188 ft. One of the embankments was 72 ft. in height. The cost was upward of $10 000 000. By the completion of the Erie Canal, the time between Buffalo and Albany was reduced from twenty to ten days.

When the Board of Canal Commissioners was created, one of the first steps taken by it was that of selecting a chief engineer. William Weston, the English engineer who had been so prominent in some of the earlier enterprises connected with canal and lock navigation in America, was urged to cross the water again and undertake the work, and was offered a salary of $10 000, but his advancing years led him to decline the offer.

With an exalted faith in the ability of their fellow-countrymen, the Commission then resolved to entrust the construction of the canal, at that time the largest and most important work of canal construction in the world, to native talent. The line was divided into three divisions, in charge of James Geddes, Benjamin Wright and Charles Brodhead, and these men, with a few able associates, took up the burden, and, with faces turned toward the future, proceeded to execute the work.

The question naturally arises—how could these men, without text books or traditions to guide them, succeed in carrying out so important an undertaking? Geddes was a judge in the county in which he resided and was 54 years of age. Wright was also a judge and had passed his 46th year. Bates and Roberts, their associates, were also judges. To understand the situation correctly we must turn backward a page. In almost every frontier settlement could be found some man, more gifted than his fellows, who laid out farms, settled petty controversies in his county and became a judge, thus uniting surveying and the execution of the law. They were generally men of courage, energy and skill, and accustomed to solving all sorts of problems. They were essentially men of action, and many of them were men whose lives had been spent in the hard school of adversity.

Benjamin Wright was born in Connecticut on October 10th, 1770. At an early age he began the work of surveying farms at Fort Stanwix, on what was then the border of civilization. Before the year 1800 it is

said he had laid out into farms more than 500 000 acres in western New York. His first work with the level was on the old Western canals, of which Mr. Weston had been the engineer. In 1803 the company decided to have some levels taken. They owned a leveling instrument, but no one could be found who knew how to use it. Judge Wright was appealed to, and after a study of the instrument and a trial, he succeeded in accomplishing the work, which led to other business for the company. What more natural than that he should have been employed in some of the preliminary work for the Erie Canal in 1811?

James Geddes was seven years the senior of Benjamin Wright. He was born in Pennsylvania on July 22d, 1763. When an appropriation of $600 was made for some of the preliminary surveys of the Erie, Geddes was selected. He made a survey from Oneida Lake to Ontario, from Lewiston to Niagara Falls, and from Buffalo to a connection with the Seneca River, all for the sum of $673, and reported that a canal from Lake Erie to the Hudson was feasible.

Canvass White, one of the junior engineers employed upon the canal, was born on September 8th, 1790, had had a fair common school and academy education, and was a man of scholarly instincts. In 1817 he left the work of construction and visited Europe for the purpose of studying canals in the old world; he traveled 2 000 miles on foot and brought back to the Erie Canal plans and instruments which were of incalculable benefit to the work. The structures built on the canal after his return were laid in cement made from rock which he discovered.

Is it at all wonderful that men such as these pushed the Erie Canal to a successful completion? They felt their way along, working out each problem, as it came, with energy and determination. What they did not understand they conquered by diligent study, unwearied zeal and sound common sense. By the constant exercise of these qualities, they laid the foundations of the profession of the civil engineer in the United States.

The action of New York in regard to the Erie Canal was the means of kindling enthusiasm in other states, and many canals were started all over the country. A large number of engineers found employment in carrying on the surveys for these works and in superintending their construction.

Besides the names already mentioned, the following engineers began

their careers on canal construction: John B. Jervis, Horatio Allen, William J. McAlpine, David S. Bates, Nathan S. Roberts, William M. Roberts, Jesse L. Williams, Major David B. Douglass, Jonathan Knight, Sylvester Welch, Ashbel Welch, Edward Miller, Samuel Forrer, James Worrall, Samuel H. Kneass, John Wilson, W. Hazell Wilson, and the list might well be extended to include others. Some of these engineers were later prominently identified with railroad enterprises, and their careers more properly belong to that era.

Canvass White, who was one of the most distinguished among the early canal engineers, became Chief Engineer of the Union Canal, and Consulting Engineer for the Schuylkill Navigation and the Delaware and Chesapeake Canal. He was Chief Engineer of the Lehigh and of the Delaware and Raritan Canals. He died in 1834 in his 44th year. With his fine abilities and character, had he been spared for a longer life, he would undoubtedly have left behind a brilliant record in the later developments of the country.

David Stanhope Bates was born June 10th, 1777. He was well educated, and was a surveyor and judge in 1810. In 1817 he began his career on the Erie Canal, and was a Division Engineer from 1818 to 1824, and the execution of some of the most important structures on that work came under his supervision. Between the years 1824 and 1829 he was in Ohio engaged in extensive works in that state, and later became Chief Engineer of the Chenango Canal and also of the Genesee Valley Canal. Judge Bates afterward became interested in railroads, and in 1835 retired, with impaired health, from the active practice of his profession. He died November 28th, 1839.

Nathan S. Roberts was born July 28th, 1776, and entered the service of the Erie Canal in 1816. He rose rapidly in his profession, and in 1828 became Chief Engineer of the Pennsylvania Canal. From 1835 to 1841 he was associated with John B. Jervis and Holmes Hutchinson in the enlargement of the Erie Canal. He died November 24th, 1852.

As a search for the fathers of the profession must, undoubtedly, be made among these early canal engineers, and it is of interest to know if among them there is one pre-eminently entitled to be designated as "The Father of the Profession," the careers of four of the most prominent have been selected for examination. They are James Geddes, Benjamin Wright, Loammi Baldwin and John B. Jervis, and their names are mentioned in the order of their birth.

In addition to his work on the surveys of the Erie Canal, James Geddes, in 1818, was appointed Chief Engineer of the Champlain Canal, and remained in charge of its construction until 1822, when he was selected to make the surveys for a canal from the Ohio River to Lake Erie, and in 1823 was called to Maine to locate a canal from Sebago pond to tide water at Westbrook. In 1827 he entered the service of the General Government on the location of the Chesapeake and Ohio Canal, and in 1828 was employed by Pennsylvania on canals in that state. He died on August 19th, 1838, at the age of seventy-five.

While Benjamin Wright was engaged upon the building of the Erie Canal, he was consulted on the Northampton, Blackstone, Chesapeake and Delaware Canals and by the State of Virginia on a proposed canal from Richmond to the Ohio. In 1824 he became Chief Engineer of the Chesapeake and Delaware Canal, and in 1825 Consulting Engineer for the Delaware and Hudson. In 1828 he became Chief Engineer of the Chesapeake and Ohio Canal, a position which he resigned in 1831. In 1832 he was appointed Street Commissioner of the City of New York, and subsequently Chief Engineer of the Hudson Railroad, and Consulting Engineer of the St. Lawrence Ship Canal, Canada, and also of the Welland Canal. Later, he was employed to survey the route for the New York and Erie Railroad, and in 1835 he was called to Havana to advise on railroad projects.. During the next four years he was engaged on several canal and railroad plans, but remained principally in Virginia, to which state he had been invited after the completion of the Erie Railroad survey. He died on August 24th, 1842, being then nearly seventy-two years of age.

Loammi Baldwin was born May 16th, 1780. He was the son of Col. Loammi Baldwin, who superintended the construction of the Middlesex Canal. The son, also a colonel, was graduated from Harvard College with the famous class of 1800. In 1807 he visited England to study engineering. One of his earliest works was the construction of Fort Strong on Noddle Island. Between the years 1817 and 1820 he was employed in Virginia on various works of internal improvement. In 1821 he was appointed Engineer of the Union Canal in Pennsylvania. In 1824 he was in Europe, and it was at this time that he laid the foundation for his large library of works on engineering. From 1827 to 1834 he was employed in designing the masonry

dry-docks at Charlestown, Mass., and at Norfolk, Va., for the Government, both of these works being carried on at the same time. It was in 1834 that he made an able and elaborate report on the proposed water supply of Boston. His death occurred June 30th, 1838, at the age of fifty-eight.

John B. Jervis was born December 14th, 1795. His education was only such as could be picked up in the common schools of a new country. In 1817 he began his engineering career as an axeman on the Erie Canal, but was soon made Resident Engineer and was advanced to several responsible positions, remaining on the work until its completion in 1825. The following is a brief summary of his remarkable career.

1825–30. Chief Engineer of the Delaware and Hudson Canal.
1830. Chief Engineer of the Albany and Schenectady Railroad, and soon afterward Chief Engineer of the Schenectady, and Saratoga Railroad.
1833. Chief Engineer of the Chenango Canal.
1836. Chief Engineer of the Croton Aqueduct.
1846. Consulting Engineer of the Cochituate Water-Works.
1847. Chief Engineer of the Hudson River Railroad.
In 1850 he visited Europe, and on his return was made Chief Engineer of the Michigan Southern and Northern Indiana Railroad.
1851. President of the Chicago and Rock Island Railroad.
1861. General Superintendent of the Pittsburg, Fort Wayne and Chicago Railroad.
1864. Chief Engineer of the Pittsburg, Fort Wayne and Chicago Railroad.
In 1868 he was made one of the first trustees of the Rome Merchant Iron Mill Company.

For more than seventy years Mr. Jervis was an active worker in his profession, and at the advanced age of eighty-three he wrote a lecture on "Industrial Economy." In December, 1881, he was summoned to New York by Isaac Newton and E. S. Chesbrough in consultation on the plans for the new Croton Aqueduct. He was then eighty-six years of age. Mr. Jervis died January 12th, 1885. Owing to his wonderfully long career, I am obliged to trespass upon the fourth period of

American engineering, beginning with the formation of the engineering societies, as Mr. Jervis became a Member of this Society in 1867 and an Honorary Member in 1868.

I leave to the wisdom of the Society the selection from these four names of the one most richly deserving the title of "The Father of his Profession in the United States."

The third period of American engineering was that beginning with the introduction of railroads in 1827. As this period has been already fully described by others, it will be but briefly alluded to, and only for the purpose of completing the connection to the founding of the engineering societies.

We have seen that the judges were engineers of the early canals, and it is also true that the majors and colonels were engineers of many of the early railroads. As there was a large demand for men of technical training, the Government allowed the graduates of West Point to enter the service of the railroads, and thus it happened that engineers with military titles, and sometimes with the organization of the brigade, could be found laying out the locations and superintending the construction of the early lines. William Gibbs McNeill, George W. Whistler and John Childe were officers in the army. On the other hand, Horatio Allen, Jonathan Knight, Benjamin H. Latrobe, Moncure Robinson and William Milnor Roberts were from civil life.

The early conceptions of Oliver Evans, John Stevens and Robert Fulton, and also the old colliery and quarry tramways were preliminary steps to the advent of the railroad. In 1825 William Strickland was sent to England by a number of public-spirited citizens of Philadelphia, and he made some interesting reports. The famous Quincy Railroad was built by Gridley Bryant in 1826, but it was only 3 miles long, and, like all the first railroads, was built as a horse railroad. It was followed in 1827 by the Mauch Chunk road, 9 miles long, connecting the mines with the Lehigh River, which, like several other tramways, was simply an adjunct to a canal. It is worthy of note that within ten years from this time there were no less than 56 railroads in active operation in the United States.

The first railroad of importance was the Baltimore and Ohio. It occupies the same position among the railroads that the Erie Canal occupies among the pioneer canals. It was incorporated in 1827, a corner stone was laid on July 4th, 1828, and the railroad was practi-

cally opened in 1830. It was completed to Harper's Ferry, 82 miles from Baltimore, in 1834, at a cost of $4 000 000. It was constructed for the use of horses as a motive power, and it was only after much experimenting that locomotives were introduced. The roadbed was formed with granite sills plated with flat iron bars, which had an uncomfortable way of curling up under the traffic, and were called "snake heads"; occasionally they would break off and come up through the floors of the cars. Although every roadbed from that time to the present has been a source of trouble and expense for maintenance, the term "permanent way" was used at an early date, and is still used to describe what is actually far from permanent in construction.

Jonathan Knight and Colonel Stephen H. Long made the first survey for the Baltimore and Ohio Railroad, and Jonathan Knight was the first Chief Engineer. He was without special education, but was a man of remarkable mental capacity. He was one of the old farmers and surveyors who worked his way to the highest station by unremitting toil and the exercise of sturdy and manly qualities. In 1842 Knight resigned his position and retired to his farm, but was afterward employed as a consulting engineer. Benjamin H. Latrobe succeeded Mr. Knight as Chief Engineer of the Baltimore and Ohio, and was among the most distinguished of the railroad engineers.

Captain John Childe, born 1802, died 1858, was a consulting engineer on the Baltimore and Ohio Railroad with Knight and Latrobe, and, like his associates, left a long record of works accomplished in various directions. His own words were, "This is a glorious world and there is much to do in it."

Horatio Allen, born 1802, died 1889, was one of the Nestors of railroad construction in this country. He was first associated with Benjamin Wright on the Chesapeake and Delaware Canal in 1824, and afterward became an assistant to John B. Jervis on the Delaware and Hudson Canal. In 1827 Mr. Jervis gave his assistant leave of absence to visit Europe, and in England Mr. Allen made the acquaintance of George Stephenson, and finally purchased three locomotives which he brought back with him. One of these, the Stourbridge Lion, he put together and ran on the Delaware and Hudson Railroad in 1829. This was the first locomotive ever tested on an American railroad.

In 1829 Mr. Allen was made Chief Engineer of the South Carolina Railroad, which he designed for a steam railroad, the first in the

world intended for locomotives, and the first long railroad built in the United States. It extended from Charleston to Augusta, Ga., 136 miles. The first hundred miles of railroad iron ever laid in one continous line was laid on this railroad.

From 1834 to 1837 Mr. Allen was abroad, and on his return was engaged as Principal Assistant Engineer on the Croton Aqueduct, and afterward became one of the Croton Commissioners. In 1842 he became one of the proprietors of the Novelty Iron Works, and was President until 1870. He was President of the New York and Erie Railroad and its Consulting Engineer. Mr. Allen was also Consulting Engineer of the Panama Railroad, and his last official position was that of Consulting Engineer of the East River Bridge. Mr. Allen became a Member of this Society in 1867, was President from 1871 to 1873, and became an Honorary Member in 1874.

One of the most distinguished of the early railroad engineers was William Gibbs McNeill, of North Carolina. He was graduated from the Military Academy in 1817. He was employed upon the Chesapeake and Ohio Canal, 1824, the Baltimore and Ohio Railroad, 1827, and the Baltimore and Susquehanna, 1830. He resigned from the army in 1837, with the rank of major by brevet. He was either Chief Engineer or Assistant Engineer of the following railroads: Paterson and Hudson, Boston and Providence, Providence and Stonington, Taunton and New Bedford, Long Island, Western of Massachusetts (now the Boston and Albany), and others. In 1842 he put down the Dorr Rebellion in Rhode Island and was made a Major-General of the militia. He died February 16th, 1853.

George W. Whistler, born May 19th, 1800, was graduated from West Point, and in 1828 was sent to England with McNeill and Knight in behalf of the Baltimore and Ohio Railroad. He was employed on several other roads, and in 1833 resigned from the army. In 1834 he became Engineer for the Proprietors of Locks and Canals, Lowell, Mass., and later aided McNeill in constructing the Western Railroad of Massachusetts. In 1842 Whistler went to Russia to build the road from St. Petersburg to Moscow, and died in that service April 7th, 1849. Whistler's fame added fresh laurels to the profession of engineering. One of his sons is the distinguished artist, James A. McNeill Whistler, of London.

Among other engineers who attained celebrity in this branch of the

profession were James E. Williams, William Milnor Roberts (a statement of whose remarkable career may be found in the records of this Society), James Laurie, Moncure Robinson, J. Edgar Thomson, James P. Kirkwood, William J. McAlpine, Ellis S. Chesbrough, and others whose names deserve mention, but their lives and works, as a rule, have been either recorded in the biographies of the Society or they encroach upon a later period of engineering.

The venerable W. Hazell Wilson, now living, was among the early engineers employed upon the Baltimore and Ohio Railroad, and he has written some of the most interesting accounts of the pioneer railroads. Mr. Wilson's father was an old canal engineer, and his grandfather and great grandfather were employed in engineering pursuits. Mr. Wilson's sons, the Wilson brothers of Philadelphia, are prominent in the profession, and their sons are now entering the field, so that we have in connection with this family of engineers, the noteworthy spectacle of six generations, in direct descent, engaged in the same profession, and five generations in this country.

No notice has been taken of the work of the bridge builders, but they should not be forgotten. As early as 1792 Timothy Palmer began to build bridges over some of the largest rivers. In 1806 he completed the Philadelphia Bridge over the Schuylkill. Mr. Weston designed the coffer-dams for the piers, which were in deep water. At about this time Theodore Burr and Louis Wernwag began to build long-span wooden bridges, in which they accomplished some remarkable work. They were followed by Squire Whipple and that meteoric genius, Charles Ellet, Jr., whose assistant, John A. Roebling, designed the grand suspension bridges so well known to the profession. An interesting historical *résumé* of the early bridges will be found in Mr. Theodore Cooper's able paper on "American Railroad Bridges," published in the *Transactions* of this Society in July, 1889.

The foregoing review has been prepared for the purpose of giving in connected form an idea of some of the most noteworthy among the earliest public works of the civil engineer, with brief outlines of the lives of the fathers of the profession in the United States.

There is much to be gained from a careful consideration of the important work accomplished by these noble and self-sacrificing men, who, in the face of almost insurmountable difficulties, successfully laid the foundations for the commanding position which

the profession now enjoys. Some of the works which they created have long since been swept away in the advance of population, others have been absorbed in the larger developments of succeeding generations, but many still survive as living monuments to their constructive skill. In any event, the heroic struggles and the masterly achievements of the early engineers are deeply engraved in the history of every undertaking which their busy brains designed or their willing hands executed. This is due largely to the disinterested nature of their services, which were directed to the welfare of the public rather than to the advancement of their own glory. Even their lives were sometimes sacrificed to the sense of duty. They sowed and others have reaped the harvest from their honest and faithful toil. No eulogy we can pronounce upon their lives will add to the luster of their fame, but it will aid us to recall the debt of gratitude we owe to our predecessors. However permanent, however useful, however grand the engineering works of the present day may be, their success is due to the failures and successes of the works that have preceded them. In these closing hours of the nineteenth century, which has witnessed the rise and progress of the civil engineer in America, we may well glance backward to gain fresh inspiration from the courage, devotion and achievements of the early founders of the profession. A just pride in their work will lead us to emulate their example and to be faithful in our day and generation to the inheritance derived from their talents and industry.

MEMOIRS OF DECEASED MEMBERS.

DAVID LEONARD BARNES, M. Am. Soc. C. E.*

DIED DECEMBER 15TH, 1896.

David Leonard Barnes was born in Smithfield, R. I., August 23d, 1858. When he was but eleven years old his father died, and the young man was led thus early in life, to the exercise, in a serious way, of self-reliance and responsible judgment. At the age of fifteen he entered a civil engineer's office in Providence, where his work included town surveys and the laying out of streets. His mind, however, was set upon obtaining an education better than that afforded by the routine of a surveyor's office, and he spent the three years following 1876 in Brown University and the Massachusetts Institute of Technology as a special student, while the vacations were occupied in engineering work. He received the degree of Master of Arts from Brown University, but took no degree from the Institute of Technology. Although surveying and municipal work were his earliest engineering employments, both the bent of his mind and his special studies inclined to the mechanical side of civil engineering, and it was in this that his reputation was won. He served three years following 1879 in the machine shops and drafting rooms of the Rhode Island Locomotive Works, at Providence, R. I., and the Hinckley Locomotive Works, Boston, Mass. He was Chief Draftsman at the Rhode Island Works when, in 1884, he left to become the Mechanical Engineer and Chief Draftsman of the New York Locomotive Works at Rome, N. Y., which were built in that year. Returning to Providence he passed three more years as Chief Draftsman and Mechanical Engineer of the locomotive works with which he had begun his mechanical practice.

Mr. Barnes had now become well known as a designer of railway rolling-stock and an authority on many other matters connected with railway operation, and in 1888 he entered upon consulting practice, with offices in Chicago, Ill., and New York City. During his service in the locomotive shops he had designed and superintended the construction of about 800 locomotives, and also a large number of stationary and steamboat boilers and special tools. At the Rhode Island works he had planned and supervised the construction of dynamos and electric motors for street railway equipment, and designed the shops of the Boston and Maine Railroad. The shops of the Bentley-Knight Electric Railroad Company were completed, fitted up, equipped

* Memoir prepared by the Secretary from papers on file and from various sources.

and put in operation under his direction. The equipment of the Allegheny, Pa., Electric Street Railway was also designed under his supervision. At the Rome works, Mr. Barnes designed, purchased and superintended the construction of a large part of the machinery installed, as well as of the first locomotives built at the works. Other work of a general character included the design, for the Providence Ship Windlass Company, of steam windlass and hoisting gear for the United States Government. In 1888 he became consulting engineer to the *Railroad Gazette*, New York City, a relation which was later broadened to engagement as mechanical editor, which connection was maintained until his death.

Mr. Barnes' work as a consulting engineer included designs, tests, inspection and reports covering the whole range of railway rolling-stock and operating devices, manufacturing plants, iron-work for buildings and steam-engine construction. He was engaged as consulting engineer by the London Central Subway Railway and by the New York Underground Railroad. The Chicago and South Side Rapid Transit Railroad, known as the Alley Elevated, retained him to have general supervision of the design and construction of its rolling equipment, signaling, lighting and shop plant. He made a radical departure from the current practice on elevated railways, by introducing, in 1893, compound locomotives on the Chicago line. When the Baldwin Locomotive Works and the Westinghouse Electric and Manufacturing Company decided to join forces in an endeavor to realize the union of the essential features of the standard steam locomotive and the accepted practice in the arrangement of mechanism for electric traction, Mr. Barnes was engaged to take charge of the work. He entered upon this labor with enthusiasm, and the designs produced have been commended for their robust simplicity and adaptability to the work required.

Mr. Barnes was endowed with unusual strength and energy, both physical and mental. He was, by nature, a mathematician, and had strong powers of analysis and reasoning. He found pleasure in the discussion of technical questions, and his arguments were marked by a directness of address to the main issues, which irrelevant details were never allowed to obscure. He had the scientific love of truth, and his published work was of a nature to stimulate his fellow engineers to thought and effort. A rapid and systematic worker, in addition to his editorial labors for the technical press, he made many contributions to the publications of the Master Car Builders' Association, the Master Mechanics' Association and the Western Railway Club, and his work included numerous reports on locomotive and car construction, and on electricity as a motive power for railroads. He carried to completion a revised edition of "Compound Locomotives," taking up the work after the death of Arthur Tannatt Woods, the author, and

about a year before his death he wrote a book on "Electric Locomotives."

Mr. Barnes became a Member of the American Society of Civil Engineers, July 2d, 1890. He was also a Member of the American Society of Mechanical Engineers, an Associate Member of the Institution of Civil Engineers, a Member of the Engineers' Club of New York and of various societies of railroad operating officers. He was married April 7th, 1896, to Ida, daughter of Colonel B. J. D. Irwin, U. S. A. (Retired), formerly Assistant Surgeon-General. In the summer of 1896 Mr. Barnes relaxed business engagements for a trip to Europe in hope of improved health, but was soon obliged to return for medical treatment. During the summer his strength steadily declined, but he did not cease work until two weeks before his death.

THOMAS SPARKS BISHOP, M. Am. Soc. C. E.*

DIED NOVEMBER 13TH, 1898.

Thomas Sparks Bishop was born March 12th, 1846, near Columbus, Burlington County, N. J. His father was a farmer, and his early educational facilities were such as were usually afforded by the ordinary country public and private schools. Before entering college he was a pupil at Westtown Boarding School near Westchester, Pa.

He entered the Philadelphia Polytechnical College September, 1864, and was graduated as Civil Engineer in June, 1866. The Master's degree was conferred upon him in 1869. Early in the succeeding August he was appointed Rodman on the engineer corps of the Camden and Amboy Railroad, and was engaged in the location and construction of the Pemberton and Hightstown, and Camden and Burlington County railroads until their completion, about a year and a half later.

In March, 1868, he was appointed Assistant Engineer of the Freehold and Squankum Marl Company's Railroad, after the location of which, in June of that year, he was recalled to the main line of the Camden and Amboy Railroad in the capacity of Assistant Engineer. In addition to his other duties there, he had charge, until January, 1872, of the location and construction of various branch railroads. During this time, however, he was detailed as Resident Engineer to take charge of the construction of the inclined plane machinery, etc., of the Bingham Mining and Lumber Company at Renovo, on the Philadelphia and Erie Railroad, where he was engaged from September to December, 1868.

In January, 1872, after the lease of the works of the United Companies by the Pennsylvania Railroad Company, he was appointed Assistant Engineer of Construction on the United Railroads of New Jersey Division, and in December of that year was assigned as engineer in charge of the improvements at Harsimus Cove and Bergen Hill, at Jersey City. Upon their completion in January, 1875, he was transferred to Trenton, N. J., where he made re-surveys of the various railroads of the United Railroads of New Jersey Division for the real estate and maintenance of way departments, until May, 1880, when he resigned to accept an appointment as engineer with the Russell & Erwin Manufacturing Company at New Britain, Conn. He served in that capacity for eighteen years, resigning his position June 1st, 1898, to devote his attention to private practice as a civil engineer and surveyor, with headquarters at New Britain, Conn.

Mr. Bishop was married in November, 1873, and was elected a Member of this Society September 2d, 1885.

* Memoir prepared by T. Sparks Bishop, Esq., from autobiographical data left by his father.

FRANCISCO JAVIER CISNEROS, M. Am. Soc. C. E.*

DIED JULY 7TH, 1898.

The name of Cisneros is an old and honored one in Spain and Cuba, each generation adding to its luster, through various channels, literature, science, or patriotism. The founder of the Cuban branch of this family was Pascual Jimenez de Cisneros, a distinguished Spanish military officer, who emigrated to Cuba in 1763, marrying there about six years later. Hilario Cisneros y Saco, the father of Francisco Javier, was a jurist whose fame for sterling integrity of character and for patriotism still endures in the memory of the Cubans.

Francisco Javier Cisneros was born at Santiago de Cuba in December, 1836, receiving his earlier education in that city. From there he went to Havana, pursuing his studies at the more advanced school of José de la Luz y Caballero, and later at the University of Havana, where he was graduated with high honors in April, 1857, receiving the degree of C. E.

His first experience was obtained as Assistant Engineer on the surveys of the railroad from La Mulata to Pinar del Rio, near the northerly coast of Cuba. In the following year, 1858, he was employed as assistant on the construction of the Sagua-la-Grande Railroad. Subsequently he was engaged on the construction of a branch of the Savanilla Railway, and on the completion of the latter he assumed charge of the construction of a portion of the Cardenas Railway. Immediately after the termination of this work, he was appointed to the position of Engineer in Charge of the Ferro-carril del Oeste, running from Havana to Pinar del Rio. He finally became General Manager of this road, which position he retained until 1867.

Always mindful of his country's welfare, and viewing the tyrannical rule of Spain with the natural repugnance of a spirit embued with the love of liberty and independence, Mr. Cisneros commenced early to use his endeavors toward the amelioration of the political evils. A party having been formed among patriotic Cubans to further liberal reforms in the government of Cuba—reforms which the more cultured class were persistently demanding, Mr. Cisneros became an active spirit therein, and in 1867 assumed the management of the periodical *El Pais*, devoting all the efforts of his caustic pen to the denunciation of Spanish misrule, and the espousal of liberty. Under the pretext of establishing agencies for the paper, he visited all parts of the island, thus directly coming in contact with the advocates of home rule and independence, and thereby laying the foundation for the Revolution of Yara, which broke out in October, 1868. From that date until

* Memoir prepared by Theodore A. Leisen, M. Am. Soc. C. E.

1871 all his energies and resources were dedicated to his country's cause. He personally arranged and conducted seven successful filibustering expeditions from the United States to Cuba, and also in the field lent his services to the struggle against Spanish rule. During this period he found opportunity to contribute many inspiriting articles to the Cuban revolutionary periodicals, and he also wrote several professional papers.

In 1871, forced through lack of consistent co-operation to desist from farther active work in behalf of Cuban independence, Mr. Cisneros decided to take up his residence in New York, and pursue his profession, at the same time declaring his intention of becoming an American citizen—a citizenship of which he was always proud. Coincident with his return to New York he, in partnership with Mr. A. G. Menocal, opened an office at 115 Broadway, and carried on a general engineering practice until his departure for South America.

About this time, when the controversy between the advocates of broad and narrow-gauge railroads was at its height, Mr. Cisneros prepared and read a paper before the New York Society of Practical Engineers, of which he was a member, strongly supporting the claim for narrow-gauge roads for new and undeveloped countries, such as the South American Republics. This article, with some additions, was published later in both English and Spanish, and attracted considerable attention, particularly in South America.

During 1871–72, Mr. Cisneros was commissioned by the Government of Peru to make a detailed inspection of the proposed Ferrocarril del Cerro de Pasco, and report thereon, and this work was performed to the entire satisfaction of that Government.

Toward the latter part of 1873, the Government of the State of Antioquia, United States of Colombia, requested Mr. Cisneros to assume charge of, and construct, the proposed railroad from Puerto Berrio, on the banks of the Magdalena River, to Medellin, since known as the Ferro-carril de Antioquia. Previous to this time narrow trails or mule roads were the sole avenues for transportation, and the railroad was designed to open up and develop the rich mining fields of Antioquia. Mr. Cisneros undertook the task assigned to him, entered into a contract with the Government of Antioquia, made the necessary reconnoissance and preliminary surveys, and commenced the construction of the road. In this work seemingly insurmountable difficulties were encountered and overcome, and in view of the existing conditions it was an accomplishment under which a man of less indomitable energy would have succumbed. Just as the first section of the road from Puerto Berrio to Pavas was completed and opened to traffic, the country was thrown into a state of revolution, and during the attendant turmoil the terms of the contract with Mr. Cisneros were ignored, and the road was seized for military purposes. Mr. Cisneros therefore refused to proceed

with the work, the contract was rescinded, and the road was turned over to the Government.

In 1877, at the request of General Julian Trujillo, President of Colombia, Mr. Cisneros agreed to assume charge of the construction of the Cauca Railway, projected from Buenaventura, on the Pacific Coast, to Cali, and in January, 1878, commenced the preliminary surveys. The actual work of construction was started later during the same year. Mr. Cisneros had in view, as an ultimate object, the joining of this with the Antioquia Railway, but this project never reached farther than the preliminary surveys. The work on the Cauca Railway was interrupted after about 30 miles had been built, by the same revolutionary movement which stopped the work in Antioquia, and, the same conditions prevailing, this contract was likewise rescinded.

In 1881, Mr. Cisneros entered into a contract with the Government of Colombia to carry on the construction of a section of the Ferro-carril de Girardot, which had been projected from the head of navigation on the Upper Magdalena River, to Bogota. During the same year, under a special arrangement with the Government, and by direct order of the National Congress, he assumed charge of the construction of the Ferro-carril de La Dorado, connecting the navigable waters of the Lower and Upper Magdalena Rivers. This road was completed from Las Yeguas to a point a few miles above Honda, after which Mr. Cisneros disposed of his interests therein.

In 1884, Mr. Cisneros, by purchase, secured control of the Ferro-carril de Bolivar, running from Barranquilla to Salgar, and shortly afterward commenced the extension thereof to its present terminus at Puerto Colombia. A syndicate, known as the Barranquilla Railway and Pier Company, Limited, was formed in London to control and operate this road, Mr. Cisneros remaining in charge as Managing Director up to the time of his death. Under his management both road and traffic were steadily built up and improved. In 1892 he constructed a steel pier 4 000 ft. long at Puerto Colombia, the ocean terminus of the road, extending out into the harbor to about 5 fathoms of water, thus enabling all ocean steamers, which formerly were compelled to lighter their cargoes ashore, to discharge directly into cars.

Mr. Cisneros was also engaged in many other important enterprises in the Republic of Colombia. He expended considerable labor in canalizing the Upper Magdalena River, deepening and clearing the channel at all the dangerous points, opening this portion of the river to steam navigation where formerly only canoes and barges were used, and doing much besides to improve navigation on the Lower Magdalena. He also established lines of steamers on the Upper and Lower Magdalena. He was chiefly instrumental in organizing the Colombian Transport Company, which operated jointly all the steamers on this river and its branches, and for a time he acted as Director General of

the company. He constructed and operated the Barranquilla Tramway, and had large interests in the Ferro-carril de Cucuta.

During the past twenty-five years, the development of Colombia has been largely influenced and accelerated by Mr. Cisneros, and it is seldom that a country's prosperity owes so much to the efforts of one man. His endeavors in that field were not toward personal aggrandizement, but for steady development of the enterprises under his management, and, consequently, an equal development of the country's resources. Ultimately, the government and people of Colombia appreciated this fact. Their confidence in him was most recently evinced by selecting him to defend their interests in a controversy between the Cauca Railway Company and the Colombian Government, which was submitted to arbitration in New York. It was due to his management of the case, and to the firm stand which he took, that the Government of Colombia was not forced to submit to one more unjust award to a foreign corporation.

Mr. Cisneros was an indefatigable worker, undaunted by difficulties or reverses, allowing no obstacles to swerve him from his pre-determined path. As a man and a friend he had no peer. With broad ideas of justice, chivalrously considerate of those associated with him, asking of no one what he was not willing to undergo himself, kind and sympathetic, he endeared himself to all who were brought into close contact with him—a man whom it was a privilege to know, an honor to call a friend.

Mr. Cisneros became a member of the American Society of Civil Engineers on May 15th, 1872, and of the Institution of Civil Engineers, London, on February 5th, 1884.

ALBERT FINK, Past-President, Am. Soc. C. E.*

DIED APRIL 3D, 1897.

It is given to few engineers to achieve greatness in more than one specialty. The profession now comprises so many branches that few can reap distinction in more than one particular. Some become eminent as investigators, or as constructors, or as managers, but the capacity to excel in all those directions is seldom found in the same man. But it can be said of the late Albert Fink, who was President of this Society from November 5th, 1879, to November 3d, 1880, that in the early part of his career he ranked as the leading bridge engineer of this country. Later he made a great reputation as an investigator of railroad economics, and as a railroad manager under exceptional difficulties; he gained still higher fame as a statesman, directing an association of railroads, and making most valuable contributions to the rational solution of the railroad problem.

The following is the bare skeleton of dates furnished by Mr. Fink himself to the "Biographical Directory of the Railway Officials of America," when asked by the *Railway Age* of Chicago for a sketch of his career:

"Born Lauterbach, Germany, October, 1827. Entered railroad service December, 1849. To October, 1853, Assistant to Engineer, Baltimore and Ohio Railroad, in charge of bridge and depot construction, west of Cumberland. October, 1853, to August, 1855, Resident Engineer, Parkersburg Branch, Baltimore and Ohio Railroad. August, 1855, to July, 1857, Division Engineer, same road; also Consulting Engineer, Norfolk and Petersburg Railroad. July, 1857, to October, 1859, Assistant Engineer, Louisville and Nashville Railroad; October, 1859, to October, 1865, Chief Engineer and Superintendent of road and machinery department. October, 1865, to July, 1870, General Superintendent, and July, 1870, to October, 1875, Vice-President and General Superintendent, same road. 1872 to 1875 also Vice-President South and North Alabama Railroad Company. October, 1875 to May, 1876, Commissioner Southern Railway and Steamship Association. June, 1877 to date (1889), Commissioner Trunk Line Association. Office, New York City."

This gives a most inadequate idea of the achievements of this distinguished engineer, and even this memoir must fall far short of fully presenting his great abilities and eminent services to his adopted country.

Albert Fink received his early education in his native town in a private school, in which boys were prepared for the university course, much attention being there paid to the study of Latin and Greek. His father was an architect, and, as Albert evinced a desire to follow this profession, he was sent to the polytechnic school at Darmstadt, where he took a course in engineering and architecture, graduating with high

* Memoir prepared by O. Chanute, Rudolph Fink and H. G. Prout, Members, Am. Soc. C. E., Committee appointed by Board of Direction.

honors in 1848. From there he went to Offenbach, near Frankfort-on-the -Main, entering the service of a contracting firm engaged in the construction of public buildings, factories and dwellings, and there he gained much practical experience in his profession. The German Revolution of 1848 in which, as an ardent lover of freedom and a firm believer in democratic institutions, he took a deep interest, proving a failure, and the consequent reaction producing a most unfavorable condition of things, he decided to emigrate to America. He returned to his home early in 1849 to prepare himself for the important step he was about to take, by thoroughly reviewing his polytechnical course and by studying the English language. He was already exceedingly systematic, and made for himself a schedule of daily studies, allowing nothing to distract him from the performance of the task he had imposed upon himself, being fully determined to omit nothing that he could do to achieve success in life.

In the spring of 1849, the year in which so many young Germans left their fatherland, he came to America. Few men were better equipped than he for the battle of life, and none had more courage to engage in it. Nevertheless, he found much difficulty in getting a start, and his courage, patience and power of endurance were severely tried before he finally succeeded in obtaining congenial employment. In New York, where he landed, he was unable to do so, but soon after his arrival in Baltimore he had the good fortune to get a position in the drafting office of Benjamin H. Latrobe, one of the first great engineers this country produced, who was the Chief Engineer of the Baltimore and Ohio Railroad, then under construction.

In those days engineers and draftsmen who could make original designs were not as plentiful as now, and such work fell almost entirely upon the chief engineer. So it was natural that Albert Fink's thorough schooling and his practical experience should cause him to soon attract Mr. Latrobe's favorable attention and cause his rapid promotion. It was not long before he became Mr. Latrobe's principal assistant and was put in charge of designing and building bridges, stations, shops, etc., for the new road between Cumberland and Wheeling, a position which he retained until the completion of the line, in 1853. During this time he produced some of his best engineering works and invented his well-known truss, which enabled him to increase considerably the length of span in iron bridges.

His first design on this plan was the bridge over the Monongahela River at Fairmont, W. Va., consisting of three spans of 205 ft. each, built in 1852, this being then the longest iron railroad bridge span in this country.

The immediate cause of the invention of this truss was the desire to avoid building many masonry piers in a large river subject to sudden rises, which would have greatly delayed the opening of the road

for traffic, and the desire to secure an iron bridge with less material than the "Bollman" truss* would have required for the same length of span.

The Fink truss is chiefly remarkable for its great simplicity. The strains in each member can be readily ascertained without intricate calculation, and they are always either compressive or tensile, there being no member subjected to both these strains alternately, while proper provision is made so that no injurious effects are produced by expansion and contraction caused by changes in temperature or by variable loads.

The Monongahela Bridge was a through bridge with cast-iron chords and posts and wrought-iron tension rods, with pin connections throughout. The ends of the chords were hung by links upon pins in cast-iron towers, and the floor beams were hung by suspension links to the pins connecting the tension rods with the posts—thus allowing all parts of the truss to expand and contract without deranging the structure. Once properly erected no subsequent adjustments were ever necessarry on such a bridge. The plan was promptly adopted by Mr. Latrobe for all the bridges on the Baltimore and Ohio Railroad and the Parkersburg Branch.

About the same time Albert Fink built two cast-iron high viaducts of novel design. They were erected upon a steep mountain grade and upon sharp curves, and were considered bold structures and were much admired. When the Society held its Annual Convention at Deer Park, in 1885, the Baltimore and Ohio Railroad officials took the members in a special train to view these viaducts, which were at that time still in good order.

Besides bridges and viaducts he furnished designs for stations, shops and engine houses, which were all characterized by appropriateness, neatness in appearance and by economy in the use of materials.

All these works made a considerable reputation for him, and when the Baltimore and Ohio Railroad was completed he might have rested on his laurels and turned his attention to the general introduction of his patent bridge-truss and to the accumulation of wealth. The time for this was most propitious, and he could readily have obtained the needful capital, for there was no probability of failure; but he was not inclined to follow such a course; he was ambitious to learn more and to perfect himself in his profession. So he accepted the position of Resident Engineer on the construction of the Parkersburg Branch, taking charge of the first seven miles from Grafton west. Mr. Latrobe

* The Bollman bridge was also designed by an employee of the B. & O. R. R. The Fink truss proved so superior that it came into extended use, and shops were built in Louisville for its manufacture. Many of the bridges on the L. & N. R. R. were of this type. In 1860 Mr. Fink was considered the leading bridge engineer of the United States. When in 1865 the North Missouri Railroad invited designs for the first bridge to be erected over the Missouri, the plans of Mr. Fink received the second prize. He had in the meantime designed, besides many other structures, the great iron bridge across the Ohio River at Louisville, which was begun in 1867 and completed in 1870.

was Chief Engineer and put him also in charge of the bridges and buildings. Here he made himself familiar with the use of the level and transit, and with graduation and masonry work, remaining from October, 1853, to August, 1855, when he was appointed Division Engineer of the Western Division of the same road, where he remained until its completion in 1857.

During this time he served also as Consulting Engineer of the Norfolk and Petersburg Railroad, then building under William Mahone, to whom he furnished the designs for the principal bridges and buildings on that line.

His business connections with Mr. Latrobe now terminated. Their relations had always been most pleasant, and they had formed a warm friendship which continued until death.

In July, 1857, Albert Fink went to the Louisville and Nashville Railroad as Assistant Engineer in charge of buildings and bridges—under George MacLeod, Chief Engineer. He was then in his thirtieth year and in the prime of life, with a capacity for work such as few men ever had. Though fully occupied with designs for the Louisville and Nashville Railroad he found time to prepare plans for rebuilding the Louisville Court House (in 1858) which were adopted, he being appointed to superintend their execution. In this work he displayed his talent and taste as an architect, and it added much to his reputation. The edifice is to-day the finest in the city, if not in the State, and the pride of the people of Louisville.

The Louisville and Nashville Railroad was opened through to Nashville in November, 1859. It was still very incomplete, it being estimated that nearly $1 000 000 was yet to be expended to make it a first-class road. In October, 1859, Mr. Fink was appointed Chief Engineer and Superintendent of the Road and Machinery departments. In 1861 the Civil War broke out, and there followed an experience in railroad operation, for four or five years, too dreadful, as we now think, ever to occur again. The line of road under Mr. Fink's charge ran directly through the region most affected by the contending armies in Kentucky and Tennessee, and was alternately in possession of the Confederate and Federal forces. The former first seized, in July, 1861, the 45 miles of the road in Tennessee, with its rolling stock, and in September further took possession of all the southerly portions of the line to within 30 miles of Louisville, leaving but 67 miles out of 268 which could be operated by the company. The Federal forces then advanced and the Confederates retreated southward, but while falling back they destroyed many of the bridges, either by burning them or blowing them up, burned the depots, the machine shops, the engine houses and the water stations. They tore up many miles of track, piled up the ties and burned them after having placed the rails on top, so that the latter were badly bent, and carried off

such of the rolling stock as was not in the ditch, so that the line was strewed with wrecks from one end to the other. Mr. Fink lost no time in waiting for the Federal forces to protect his workmen. He followed up the retreating Confederates with tireless energy, reconstructed the damaged portions of the road, trestled the bridges,* built temporary water stations, straightened the rails and relaid the track, and picked up the wrecked engines and cars. This work was seriously interrupted by local freshets, which washed out some of the trestles, sometimes more than once, but, to the amazement of the military, these reconstructions were completed nearly as fast as the Federals advanced, and not infrequently ahead of them. So adequate was this reconstruction and the subsequent operation that the Louisville and Nashville was the only road in the South of which the Federal authorities did not take military possession, having realized that it could be repaired and operated by Mr. Fink more efficiently than by themselves.

The Louisville and Nashville Railroad was the base of supplies for the Federal forces. Over that line all supplies had to be hauled, and it became an important object for the Confederates to destroy or cripple it. Then ensued a series of many raids, which became celebrated as those of General John N. Morgan. This dashing cavalry officer slipped through the Federal forces many times, made his way north to the line of the road, captured trains and burned them, destroyed bridges and water stations, and repeatedly stopped entirely the operations of the road. By that time Mr. Fink had organized, with great forecast, a special corps of bridge builders, bold, sturdy fellows, with which he repaired the damages as soon as the raiding forces withdrew. Indeed, in some cases, too soon, for the raiders turned back upon their tracks, drove off the repair gangs and occasionally killed some of the men. A number of times Mr. Fink's own life was in danger. Some bridges were burned three times—a tunnel was caused to cave in by burning the timbering; trains were wrecked, and indeed every class of injury was done which could be inflicted on a railroad. This continued for about three years, and was followed by minor raids of predatory bands and guerillas, there being no less than 8 detailed in the report of 1864, and 22 in the report of 1865, in which trains were captured, depots, cars, bridges and trestles were burned, and a number of the employees killed, the total amount of physical damages being estimated by Mr. Fink (report of 1866) at $689 512.39, without including the loss of traffic or the increased operating expenses. These damages were repaired with wonderful speed and vigor, although complicated by an unprecedented storm in 1865, which washed away many embankments and bridges, and stopped the operations of the road for a week. Moreover, Mr. Fink carried forward, as opportunity and means

* One of them was an iron structure 1 000 ft. long and 115 ft. high, two spans of which had been blown up, while another was 520 ft. long and 92 ft. high.

served, the work of completing the construction of many works left unfinished at the first opening of the road, so that it was in prime condition at the end of the Civil War, and did a large and profitable business as a consequence of this wise forecast.

In his annual reports, which were models of their kind, and soon overshadowed those of the directors and of the operating department, Mr. Fink dispassionately recited the destructions and misfortunes of the preceding fiscal year, together with detailed estimates of the cost of the damages inflicted. He confined himself to a calm statement of the facts, and a characteristic foot-note* in the report of July 1st, 1863, added just as it went to the printer, exhibits with what fortitude he neither allowed himself to be cast down by fresh misfortunes, nor elated by good luck.

Amid these vicissitudes he inaugurated and maintained the admirable system of accounts and records which enabled him to determine the cost of every item of work, and which subsequently attracted so much attention from railroad men. He was made General Superintendent (in addition to his duties as Chief Engineer) in 1865, and every report of the president and directors drew special attention to his efficiency and good management.

With the return of peace, Mr. Fink devoted his great energies to the reorganization and economical operation of the road. Not only did he repair the damages inflicted during the war, complete the temporary works of the original construction, and put the line into high condition, but he also made the first thorough investigation ever carried out in this country into the cost of railroad traffic, and the steps to be taken to produce the best results. Trained to accuracy as an engineer, he investigated the expense, through his system of accounts, of the various particulars of the traffic, and introduced science in making the tariffs. Up to that time railroads had been operated mostly by rule of thumb, and little attention had been paid to the actual cost of various classes or items of the traffic. The Louisville and Nashville Railroad in 1866 consisted of a main stem of 185 miles and three branches of 85, 46 and 17 miles, respectively. On these the cost of the same traffic differed very greatly, in consequence of its volume and of the local circumstances. Mr. Fink investigated the causes and remedies for this difference with great acumen. This was done by classifying the operating expenses under some seventy-five different headings, by making up statements of the amount and character of the freight and passenger traffic, and by reducing the accounts

* That foot-note was as follows: "Since then Morgan has visited the road once more. On the 4th of July he captured Lebanon, completely demolished the company's depot buildings and engine house at that place, burned all the cars standing there, and also Hardin's Creek bridge, 2½ miles this side of Lebanon. On the 6th he crossed the Main Stem at Bardstown Junction, on his way north, burning Long Lick water station, and damaging the adjacent bridge. He has not returned since (October 1st, 1863) being now confined a prisoner of war in Columbus, Ohio. Table XXII shows the damage done to the road, from July 1st to October 1st, 1863."

to one common measure per mile of road and per train mile. His annual reports became bulky pamphlets, consisting largely of tabular statements, from which sound deductions could be drawn. They were in the nature of a revelation to other railroad managers and experts. In these reports, Mr. Fink not only gave in detail the results of the operations during the preceding year, and showed how further economies might be attained, but he also discussed those larger politics of railroad management relating to tariffs, alliances, and extensions, which, when carried out by him, subsequently made the Louisville and Nashville the great system which it has become, in spite of the fact that its geographical position scarcely seemed to warrant that result in the beginning. Under his advice the road was extended to Montgomery, Ala., by acquiring the charter of the South and North Alabama Railroad Company, and upon its completion in October, 1872, he became Vice-President of that company, having held a like position on the Louisville and Nashville since July, 1870.

In reading Mr. Fink's annual reports one is impressed with his absolute candor. He never warps the facts or the argument, and he tells all there is to be told. He points out what increased earnings are due to temporary causes, such, for instance, as the increased southern traffic just after the war, before the other lines in the South could be placed in condition to compete. He shows so convincingly the unreasonableness of the chronic complaint of some local shippers that their rates were out of proportion to the through tariff, as to have put an end to that class of fault-finding for a time. He is the first to show conclusively by statistics that a branch line, unprofitable in itself, may yield an adequate profit when the increased traffic of the main line owning its securities is taken into account, and he discusses in a masterly way some of the general features of the railroad problem, such as through and local rates, effect of water competition, etc., etc.

All this was done amid the engrossing cares of reorganizing, managing and rebuilding a railroad in a region where the lawless element was still active, for in 1866 and 1867 various outrages were committed, such as the wrecking and robbery of the pay car, the wrecking and plundering of a passenger train, the robbing of a depot agent, etc. Mr. Fink prosecuted the perpetrators with great vigor. Most of the robbers were discovered and arrested. Two of them who had killed a comrade for divulging were executed, and a number of the others were sent to the penitentiary.

Mr. Fink's reports attracted such general interest among railroad men that copious extracts from the report of 1873–74 were reprinted for general circulation under the title of "Cost of Railroad Transportation." This was followed in 1874 by a treatise entitled "Investigation into the Cost of Passenger Traffic," based upon the traffic of the Louisville and Nashville Railroad since 1867, which was to have been

followed up by a second part containing the results of an investigation into the cost of freight traffic, the completion of which was prevented by Mr. Fink's official duties. The portion thus published discussed with conciseness the causes which affect the cost of transportation, the expenses connected with passenger trains, the character of the loads carried by them, with the cost of each class, the effect of an increase of business upon the cost, and the cost of mail and postal service. So able was it all that another edition was called for in 1876, and Mr. Fink was invited to appear before the special commission of Congress on "Railway Mail Transportation." The pamphlet, which was reviewed at length in the *Railroad Gazette* of May 30th, 1874, remains a classic to this day, and the full and convincing work of this distinguished engineer is still almost the only known treatise upon the abstruse subject of the cost of railroad transportation.

These continuous and absorbing labors doubtless induced in Mr. Fink a certain sense of fatigue; moreover, he desired to complete his studies of the cost of freight traffic, which traffic he believed to be more important and complicated than that done on passenger trains, and he, therefore, declined a re-election as Vice-President and General Superintendent at the annual meeting of the stockholders of the Louisville and Nashville Railroad in October, 1875. Very complimentary resolutions (published in the annual report) were passed by the directors of the road, and he began preparations for a long European trip. He was not suffered thus to get out of harness, however, for the "Southern Railway and Steamship Association," already in existence, was then looking about for a commissioner, and persuaded him to take the place until matters could be put into smooth working order.

In 1875, Mr. Fink entered on the work for which, consciously and unconsciously, he had been preparing for years. It was a work of great originality, and of difficulties quite as great. A critical period in the history of the railroads of the United States had been reached, and, in the decade between 1870 and 1880, a very important change took place in the attitude of those who controlled railroad properties toward the owners of those properties, toward other railroads, toward society and toward the State. In bringing about this change Mr. Fink had more to do than any other man, perhaps more than any other dozen men.

Very briefly stated, the change was something like this: The stage of expansion began to give way to the stage of organization. The unscrupulous, the "smart," and the corrupt railroad officers began to sink in relative numbers and power, and the men of high professional spirit became relatively more numerous and more influential. The theory that railroads were merely private properties, worked only for the profit of owners, or often for the personal gain of officers, began to give way to the theory that railroads are quasi-public institutions

enjoying valuable powers and privileges received from the State, and having corresponding duties and obligations to the State. The theory of the control of rates and service by competition broke down and began to give place to the theory of control by combination. It must not be supposed that these changes came about all at once, or that they are yet complete, or that they have gone on during the last twenty-five years without interruption; but the tendencies to these changes began to be strongly marked at this period in Mr. Fink's career.

The Southern Railway and Steamship Association was organized in 1875. It was a combination of about twenty-five southern railroads, formed for the purpose of controlling and regulating rates, to the end that they might be kept reasonally permanent and so controlled as to lead to the least possible discrimination between persons or places. It requires slight familiarity with the social economics of railroads to appreciate that such regulation of rates would sweep away most of the evils felt by the railroads themselves and by the communities which they served. The Southern Railway and Steamship Association was the first really important and influential combination of this kind, and one of the most successful ever established. It was largely of Mr. Fink's creation and was a highly organized institution. It was designed as a machine to enable the railroads to work together with or without pooling, as might be thought expedient. But the association was much more than a pool. While it was designed to divide the competitive traffic, it was designed further to facilitate handling the through traffic. To that end it arranged classifications and rates, and it served as a clearing house for through-traffic accounts. Of this great organization Mr. Fink was the working chief, and all who are able to judge agree that its affairs were well managed from the start. The plan of work and the body of rules were prepared by Mr. Fink with German painstaking and thoroughness and were administered by him with distinguished ability, justice, tact and resolution.

In May, 1876, Mr. Fink felt that he could withdraw to resume his plan of rest and study, and after about a year spent in Louisville to arrange his personal affairs, he started in June, 1877, for a trip to Europe. This time he got as far as New York, arriving there at a time when the chief executive officers of the four eastern trunk lines were endeavoring to compass measures to prevent the recurrence of the disastrous railroad war of the preceding year. Mr. Fink was consulted, and so greatly impressed the executive officers that they prevailed upon him to organize and to conduct the affairs of their proposed association, upon a plan chiefly devised by himself. The Trunk Line Association which was then formed included the Baltimore and Ohio, the Pennsylvania Railroad, the Erie and the New York Central and Hudson River Railroad. It was established to divide the west-bound traffic. Late in 1878 these lines and their western connections, north of the

Ohio River, and east of the Mississippi River and to the Atlantic, and including also the Louisville and Nashville, agreed to co-operate in all matters relating to competitive traffic, and the Joint Executive Committee was formed. Of this Mr. Fink was made Chairman, retaining also his commissionership of the Trunk Line Association. He thus became the executive officer of all the most powerful railroads in the United States, so far as concerned their competitive traffic arrangements. That he occupied a position of great power and great delicacy no one will deny. That he filled it with remarkable skill everyone will admit. Mr. Charles Francis Adams said:

"It is safe to say that the greatest of all these combinations, that of the Trunk Lines, is held together only by the personal influence and the force of character of one man, its Commissioner, Colonel Fink."

With numerous troubles and vicissitudes the Trunk Line Association was held together, and in 1898 it is still in existence, but its power and capacity were much abridged by the Federal Inter-State Commerce Act which went into effect in 1887. Mr. Fink resigned in the middle of 1889, partly for the reason that Federal legislation nullified to some extent the effect of his labors and partly for the reason that his health was failing. Indeed, for two or three years before his resignation he had been absent from his office a great deal.

The Joint Executive Committee gave place to the Joint Traffic Association which was so organized, it was believed, as to conform to the Inter-State Commerce Law and to aid, rather than to retard, the operations of that law. The Southern Railway and Steamship Association also underwent vital reorganization after the passage of the act. Many other associations of much the same character as these have been formed and still live, but on all of them Mr. Fink's influence is felt as a lasting force.

During Mr. Fink's long-continued and far-reaching labors in this great field he was associated with and aided by many able men, students or administrators, or both, but among them all he stood pre-eminent. In the establishment and growth of correct ideas of rate-making and classification he was an original student, investigator and teacher. In the development of methods of organized co-operation among railroads he was a leader. In discovering and formulating the true theories of the relations of railroads to each other and to their patrons and to the State he was one of the earliest of the really scientific philosophers.

Mr. Fink's mind was broad and comprehensive, but it was also stored with detailed information. He had a solid and clear knowledge of his subject acquired in his own way, at first hand, and thus he commanded his knowledge. His analyses of the cost of transportation, begun on the Louisville and Nashville, were the most complete that had appeared up to that time, perhaps the most complete ever made, and they have served as a basis for much of the work of other students

along these lines. Professor Hadley says that Mr. Fink's Report on Transportation Rates to the Seaboard

"Is one of the most successful applications ever made of mathematical methods to social phenomena. It ought to give Mr. Fink as high a rank among scientific investigators as he holds among practical men."

In Mr. Fink's dealings with the problems of railroad management and of their relations to each other and to the people of the nation, he did not permit himself to be guided by theories which he had evolved. He knew no such easy process. He was a true Baconian. He learned the facts as far as in him lay (and his powers of acquisition were great), and his theories grew from such knowledge, but under it all lay the clear, just spirit which could detect the real and the false and which was always directed toward an ideal justice. In one of the United States Senate inquiries by special committee, Senator Harris asked Mr. Fink if he would be willing to formulate, in the shape of a bill, such legislation as he would recommend. Senator Harris said: "I make this suggestion to Mr. Fink because I know of no man in the whole country who has more extensive and accurate information upon the railroad side of this question." Mr. Fink promptly added: "And upon the other side, too; I am looking upon the other side as well as upon the railroad side."

Mr. Fink was as far as possible from socialism. He believed in the free play of the forces that emperors or legislators cannot control, the forces which really make human progress. He always contended for the common law which embodies the wisdom and experience of past ages and which cannot be improved by special legislation; and Mr. Fink's labors if fully accepted would have made most special railroad legislation unnecessary, or would have so directed it that it would have been more efficient and beneficial. Thus he was a man whom Germany could ill spare, and whose teachings and example have helped to make our own nation strong and free.

At the first meeting of the officers of roads in the Joint Traffic Association, held after Mr. Fink's death, Mr. George R. Blanchard, Commissioner of the Association, and who, as an officer of the Erie Railway, had taken a prominent part in the establishment of the Trunk Line Association, of which Mr. Fink became the head, presented the following minute, which was unanimously adopted and ordered to be recorded, both by the managers and by the Board of Control.*

"Albert Fink, born at Lauterbach, Germany, October 27th, 1827, and who died April 3d, 1897, was from June, 1877, to July, 1889, the Commissioner of the Trunk Line Association, the Chairman of the Joint Committee, the Secretary of the Presidents, and in many instances their arbitrator. Being chosen because of his prior demonstrated fitness for those duties, he brought to their performance a thorough education and a rare combination of organizing, executive

* *Railroad Gazette*, April 16th, 1897.

and judicial faculties and ability; exercised his complex duties with eminent fairness, and with just consideration for mutual rights, and left the association voluntarily, bearing the esteem and regrets of all his associates and competitors.

"In the conduct of his duties he showed rare powers of concentration, analysis and statement, a comprehensive mind, alive to the development of the railway system and the public business and their mutual relations, the intricacy of the problems which he confronted, a thorough devotion to the task of solving the great difficulties encountered, and exceptional courage and discretion. His decisions were rarely appealed from, and most often to himself. Even when he was most characterized by the public as an autocrat in transportation rates and rules, he was giving the wisest and most liberal consideration to their interests. If the great duty which he had undertaken—to harmonize the extended rival interests which he served—was not accomplished, it was through no fault of his devotion or administration; and the expressions of his own regrets, when he left his duties, that he had not entirely achieved his purposes, will be remembered by all as a tribute to his sincerity, and should afford an incentive to continued effort to the same great ends on the part of those who now administer the same properties."

When, after this strenuous business life, the imperative need of rest came to him, Mr. Fink resigned the office of Trunk Line Commissioner, in 1889, and went for a few months to Europe; thence returning with his only child, a daughter, to Louisville, Ky., he reopened the old home where he had spent the short period of his happy married life with Sarah Hunt. He kept it as a *reste à pied*, traveling extensively, both in this country and in Europe. Scenery was a source of great enjoyment to him, and he was interested in everything great and small, from the social problems of the places he visited to their local coloring and customs. While at home he occupied himself with his books, spending the better part of every day and far into the night reading with intense interest works on philosophy and history. In his retirement he missed greatly his active business life, but, although he had many tempting offers to return to it, he declined them all.

During the last two years of his life he had severe attacks of illness caused by kidney trouble, each one sapping more his strength, both physical and mental, until his death, which came suddenly and without pain early in April, 1897.

Albert Fink's chief characteristics were conscientiousness, passionate love of truth, fine sense of justice, perseverance, untiring energy, dauntless courage and strong will. Nothing could make him deviate from his path of duty. His strong analytical mind never rested until it had found the truth, the bed-rock of facts, and had discovered all the reasons why things were so and could not be otherwise. His decisions on questions submitted to him for arbitration were nearly always correct, and were readily accepted by both sides. His tenacity of purpose, his uncommon intellectual powers, and the power of concentration, which he possessed to a remarkable degree,

enabled him to solve intricate problems in fields of labor for which he had not prepared himself, and in which he had but little experience. As an energetic worker he had, perhaps, no equal among his contemporaries. Courageously and unswervingly he advanced to the goal he had set for himself in his early youth; to educate himself thoroughly, to develop his faculties to the widest extent and so rank with the highest in his profession. When he undertook work or study he gave all his energy to it, and he did it thoroughly. He was deeply versed in philosophy and history, and interested in science, literature and art, and in touch with all the questions of the day.

His fairness, honesty and integrity made him admired by the men with whom he came in business contact. In his business dealings, he was not only straight from a business point of view, but he was morally straight. Several of his inventions made the fortunes of other men, he himself neglecting to exploit these fruitful sources of wealth. The utility and perfection of his work were his goal, not the amassing of a fortune. His employees were devoted to him; they cheerfully performed the duties he assigned them and took the deepest interest in his success. Even now, after so many years have passed away, they refer with pleasure to the fact that they once worked under him.

Albert Fink was distinguished in appearance and unusually handsome. He was tall and well proportioned. He had a noble head and broad forehead. His mouth and chin showed the strength and force of his character, while the expression of his large, dark eyes was wise, calm and thoughtful and infinitely tender. When he spoke, what he said was clear, concise and to the point; in a very few words he told a great deal.

He enjoyed society, but his life was too busy for him often to be able to indulge in social pleasure. His manners were simple and very cordial. The strength and simplicity of his character and the kindness and gentleness of his nature won for him the affection of men and women, young and old. He had a strong and tender love for little children. In his tastes he was simple, and he spent little for his personal uses, but he was lavish in giving. No one ever called upon him in vain for assistance, and, without being called upon, he lightened the burdens of many who will remember him with grateful hearts. To be able to do good, to relieve suffering, was his principal happiness. He was a most remarkable man, a true gentleman, and one who had reached the highest type of humanity.

Mr. Fink was elected a member of the American Society of Civil Engineers on July 20th, 1870; he became a Fellow on September 3d 1872. He served as Vice-President of the Society from November 7th, 1877, to November 5th, 1879, and as President from November 5th, 1879, to November 3d, 1880.

SAMUEL KILLEBREW, M. Am. Soc. C. E.*

DIED JANUARY 9TH, 1899.

Samuel Killebrew, son of George W. and Mary A. Killebrew, was born near Oak Grove, Christian County, Ky., May 14th, 1840. From early childhood he manifested a taste for mathematics, which his parents encouraged and cultivated, placing him under the care of the best teachers in the vicinity of their home and giving him a thorough preparation for college. He entered college at Russellville, Ky., in January, 1856, and remained two years, leaving, to assume his first work in the line of his chosen profession.

He began his career in engineering as rodman under Col. George B. Fleece, Chief Engineer of the old Memphis, Clarksville and Louisville Railroad in 1859. Mr. John B. Tapscott, who was in charge of the division on which he was employed, says of him:

"He was a country boy of rough exterior and looked as if he could manage a plow team or split a rail better than he could manipulate a transit or level. In less than two weeks I found my country boy was far from being a bumpkin, but was a close, observing, accurate and valuable assistant."

He continued in the work until the Civil War began, when he left the railroad to serve the Confederate Government. In 1870 to 1878 he was employed in various capacities in the location and construction of the Brownsville and Ohio Railroad in Tennessee, his continued advancement from rodman on preliminary survey to responsible charge of construction bearing evidence of his energy and ability.

In 1881 he was appointed Assistant Engineer of the Galveston, Harrisburg and San Antonio Railway, then being built from San Antonio, Texas, westward to a connection with the Southern Pacific, and was assigned to duty as instrument man in a locating party. It was on this work that the writer first met him, and here began a friendship which only terminated with his death. As is often the case when a new man joins a party that has been in the field for some time, Mr. Killebrew was not welcomed with much cordiality by some of the members of the party. On one occasion, the engineer in charge being on reconnoissance and Mr. Killebrew being in charge, there was an open revolt against his orders. The manner in which he handled this trouble showed a firmness that would not permit trifling, but withal a kindly spirit of forgiveness that won for him the good will and respect of all the party. Not long after this "the Major" was the most popular man in the party and he was never troubled again.

At this time it was found very difficult to get a location over a certain summit, with the adopted gradient, and while the party was

* Memoir prepared by E. B. Cushing, M. Am. Soc. C. E.

on this work, Major Killebrew was detached from the regular corps to assist in reconnoissance. With a corps of only two men, and with only a surveyor's compass and chain he worked out a line that was located and built upon. This was the opportunity that demonstrated his ability, and Major Converse, the Chief Engineer, gave him charge of a party on another location. During the year that followed he located a line to Fredericksburg, Texas, 70 miles, and from La Grange to Bastrop, Texas, 35 miles. Though the company for which these lines were located did not build them, yet other corporations have since constructed the roads. Notwithstanding they had the best engineering talent, their constructed line does not vary appreciably from Mr. Killebrew's stakes.

In 1882, when Mr. C. P. Huntington and his associates decided to build into Mexico, Major Converse was asked to recommend a man to locate the line. He sent Mr. Killebrew to New York, and Mr. Huntington, with his good judgment of human nature, at once employed him. This may be called Mr. Killebrew's life work and his monument. Beginning at Piedras Negras, Mexico, in the middle of a hot summer to locate a line through a foreign country, the inhabitants of which were not generally friendly to "Gringo" intrusion, the language, customs and laws being entirely new, and the water, food and forage supplies being meager, he was beset from the first with difficulties that were extremely trying. With no precedents to guide him and with imperfect and unsatisfactory maps to indicate his course, he started his work with the same confidence and cheerfulness he had displayed in former undertakings. Mounting his mustang pony, his little compass and hand level in his saddle bags, he bade us farewell and started for the mountains. When, in ten days, he returned, the general line of the Mexican International Railroad, over the first mountain pass, as it is to-day operated, was determined.

This line, as constructed, has several mountain summits to overcome and stream crossings to make, the detailed location of which involved some prompt decisions on general points, as well as nice adjustments as to details. In addition to this, the location was made through a country where adequate food and water supply was wanting. There were frequent depredations by roving bands of Indians or by natives disguised as Indians, who were worse than the originals. The question of properly maintaining a party was often as trying as the work itself. During all this time Mr. Killebrew never once lost his control of the situation, but anticipated and provided for probable emergencies, spending much of his own time long distances ahead, looking out the way, and between the preliminary corps and the locating party, which were generally 25 to 40 miles apart. His presence around the camp fire encouraged the men, and his wonderful grasp and memory of topography rendered easy the work of his

assistant engineers. He never permitted a party to leave a district between two controlling points until both the assistant engineer in charge and himself were satisfied that the line in detail was where it should be and no considerable improvement was possible.

On one occasion, the owners of the property and their consulting engineer were anxious to build the road as close as possible to a certain town. Mr. Killebrew abandoned all work ahead and ordered both his corps to the work of modifying the line, assigning one party to each side of the river and directing both in person. When the final determination was made, it was after every available route had been tried. The line, however, was not satisfactory, and as the construction advanced, other engineers were detailed to get one nearer in. Major Killebrew heard of this, but instead of being hurt, he said: "I hope they will get what is wanted, but I am afraid they will not." The road was built on the Killebrew line.

He remained with this company until all its projects had been explored and much of the line built. In 1889 he was employed by the Mexican Pacific Railway in a very difficult location on the Atlantic side of the mountains, in Southern Mexico. This location led to the head of navigation on the Grijalva River, at San Juan Bautista. A portion extended through a mountain range entirely unknown, and as the coast was approached, the route lay through swamps and a dense tropical jungle, in which life was made a burden by the swarms of stinging insects and the dangers of bites from venomous reptiles. His principal assistant, Arthur C. Wheatley, Assoc. M. Am. Soc. C. E., writing of this work, says: "He was always cheerful and encouraging in spite of the dangers, difficulties and sickness which surrounded us." On the completion of this work he returned to his home in Brownsville, Tenn.

When the Mexican International decided on further extensions, Mr. Killebrew was again sent for and placed in charge of the surveys. He made some extensive reconnoissances and surveys, including lines from Durango to Guadalajara, Durango to Guanacevi, Durango to Mazatlan, and many others; his work for this company embracing, probably, between 2 500 and 3 000 kilometers of final location, much of which was built. He was engaged in surveying a line from Durango to the Pacific Ocean when the final sickness came on him, and he died in Durango, Mexico, January 9th, 1899.

A chronological statement of his work does not constitute a proper memoir of Samuel Killebrew, for he was an extraordinary man in many respects. He was very simple in his tastes and very approachable. He was genial in manner and would go out of his way to show attention to the people living along the line of his surveys. His judgment of topography was a natural gift, but one that he always cultivated. He could ride through a new country and estimate with a wonderful

degree of accuracy the differences in level and distance between the controlling points. In later years, his health was broken, but his energy prior to this was ceaseless. He never let anything divert him from his work, and he instilled in those around him such interest in the work in hand that it was a topic of general conversation around the camp fire at night. In this way, the work was made as pleasant as possible to those who had to bear the burdens. He was painstaking with his men, never losing interest in showing them how to solve the problems that came up daily on such work, and many an engineer will, with the writer, always feel grateful for his kindly interest and assistance in those days of early trials and hardships. He was liberal with his men, and even in early days, when working on a small salary, he would share his earnings with those in need as cheerfully as when later on he could better afford to do so.

He was loyal and faithful to the interests of his employers, and had the work been his own, he could not have given it more attention. As a man, he never outgrew his boyhood's reverence and affection for his parents, and often on trips he would speak with the greatest affection of his mother and tell anecdotes of his early training. His wife accompanied him on the survey whenever it was practicable for her to do so, and was with him when he died.

His work is a monument to his professional skill, and he has left in the hearts of those who were with him tender memories of his many acts of kindness which they will never forget.

Mr. Killebrew was elected a Member of the American Society of Civil Engineers, September 2d, 1885.

FREDERICK HENRY SMITH, M. Am. Soc. C. E.*

DIED DECEMBER 24TH, 1898.

Frederick Henry Smith was born in Pittsburg, Pa., on November 10th, 1839, and there his early schooling was effected. At the age of thirteen he went with his uncle, Col. James Shaler, to Lake Superior, where, as a boy, in immediate contact with some of the great mining industries, he acquired his taste for geology and mineralogy. When but sixteen years of age he entered the engineering service of the Louisville and Nashville Railroad, under the late George MacLeod, C. E., and continued in this field until the breaking out of the Civil War, when he became a military engineer in the service of the Confederate States.

Here, besides serving on strictly military duty, he was detailed to the Niter and Mining Bureau, where he devoted his energy to the production of saltpeter and other material required for munitions of war. The caves of Alabama, Georgia and Tennessee were ransacked, under his guidance, for niter to supply the powder-mills. He was also engaged during the war in rehabilitating some of the old and disused iron-smelting furnaces, and in other metallurgical work.

At the close of the war, Captain Smith turned his varied experience to the practice of civil engineering, and, establishing himself at Baltimore, Md., became a member of The Baltimore Bridge Company. This connection was terminated in 1873, Captain Smith continuing his practice at Baltimore, and devoting much of his time to the design and erection of bridges. He became Engineer of Bridges to the City of Baltimore, and Consulting Engineer for the railroads of the Seaboard Air Line. In both these capacities he executed some excellent examples, among others the handsome stone arched North Avenue Viaduct, in the City of Baltimore, which is likely long to remain as a monument to his professional ability.

His interest in geology and mineralogy led him to employ such brief intervals of leisure as came to him in the preparation of a popular hand-book, entitled "Pocket Geologist and Book of Minerals," which he published in 1877. This was revised and republished in 1882 as "Rocks, Minerals and Stocks." In 1890, he published the "Pocket Geologist and Mineralogist," and in 1895, "Smith's Hand-Book of Valuable Minerals: What They Are; What They Look Like; Where To Look for Them," a duodecimo of 238 pages. These small volumes are full of useful and practical information for the prospector and miner, and contain in pocket form much valuable matter in relation to rocks and minerals.

* Memoir prepared by Mendes Cohen and C. H. Latrobe, Members, Am. Soc. C. E.

During the last year of his life, he was the engineer member of the State Board of Health of Maryland, where his general knowledge and experience made him an efficient member, whose opinions were highly valued by his colleagues.

Mr. Smith's health broke down completely several years before his death, but notwithstanding constant suffering, his fortitude, energy and patient good humor, sustained him to the last. He became a Member of the American Society of Civil Engineers on February 21st, 1872, and although he never took a very active part in the deliberations of the Society, yet he was always esteemed a valued member. He married on December 27th, 1870, Miss Elizabeth Gairdner, of Augusta, Ga., who, with two sons and one daughter, survives him. He passed away quietly and peacefully, much regretted by those who knew him best.

WILLIAM GARNETT WILLIAMSON, M. Am. Soc. C. E.*

DIED AUGUST 2D, 1898.

William Garnett Williamson, eldest son of General Thomas H. and Louisa Garnett Williamson, was born in Norfolk, Va., December 13th, 1840. He attended Fuller's Classical School, of Lexington, Va., and then entered Washington College (now Washington and Lee University), Lexington, Va., from which he was graduated in 1859, with the degree of B. A. He was afterward privately instructed in Civil Engineering by his father, who, for nearly fifty years, was Professor of Engineering at the Virginia Military Institute. His first service as an engineer was with General Robert E. Rodes upon railroads in Alabama. At the breaking out of the Civil War he volunteered in the Confederate Army as a private in the college company, but was soon transferred to the Rockbridge Artillery, and served with that battery under Captain, afterward General, W. N. Pendleton until he was appointed a Lieutenant of Engineers and assigned to the command of General Stonewall Jackson. Captain Pendleton was the Rector of Grace Church, Lexington, and some of Mr. Williamson's earlier military service was thus with his friend, his pastor and the instructor of his boyhood. As First Lieutenant of Engineers, Mr. Williamson was assigned to the First Regiment of Engineer Troops, C. S. A. (commanded by Colonel T. M. R. Talcott), of the Army of Northern Virginia. He was subsequently promoted to the Captaincy of Company F of that regiment, and served with it until the surrender at Appomattox Court House, where he was paroled.

After the war Mr. Williamson went to Mexico and there engaged in the construction of the railroad between Vera Cruz and the City of Mexico, of which Colonel Andrew Talcott, father of his regimental commander, was the Chief Engineer. Returning to Virginia in 1867 he was for six years engaged in railroad work in Virginia, North Carolina and Kentucky. From 1871 to 1873 he was in charge of the construction of shops, depots and yards, for the Chesapeake and Ohio Railroad, at Huntington, W. Va., and was Division Engineer of the road. From 1874 to 1876 he was Assistant Engineer on the Tennessee River surveys. Returning to railroad work, a year was passed in the service of the Buchanan and Clifton Forge Railroad in Virginia, as Division Engineer, and in 1878 he was again in the Government service as Assistant Engineer on survey work in Currituck Sound, N. C., the Nattoway, Chowan and Tar Rivers. In 1879–1880, he was with the Richmond and Danville (now the Southern) Railroad as Engineer of Bridges, under Colonel Henry T. Douglas, M. Am. Soc. C. E., with whom he was associated for several years. From 1880 to 1882 he was Assistant Engineer with the late General Quincy A. Gillmore. M. Am.

* Memoir prepared by the Secretary from information furnished by W. E. Cutshaw, William Cain and Geo. G. Earl, Members, Am. Soc. C. E., and from papers on file.

Soc. C. E., in the United States Survey of the Altamaha and Canoochee Rivers in Georgia, and later was Assistant Engineer on the survey of the Cahaba Coal Field, in Alabama.

He next undertook the construction of a sewerage system for Montgomery, Ala., from the plans of Rudolph Hering, M. Am. Soc. C. E. After the completion of these sewers, in 1883, he was elected City Engineer of Montgomery, and inaugurated a plan of improvements for that city. The work of a city engineer did not suit his sensitive nature, and in 1888 he resigned this position and again entered the government service as United States Civil Assistant Engineer on the Alabama River under Captain Hoxie, where he remained about one year. In 1888 he opened an office in Montgomery, Ala., for general engineering work, in which venture George G. Earl, M. Am. Soc. C. E., was associated with him. The firm continued for only about two years, during which time sewer design and construction and water-works extension for Americus, Ga., and sewerage plans for Columbus, Ga., were undertaken. Mr. Williamson had other work in hand at the same time, including sewerage plans for Sheffield, Ala., and the improvement of the grounds of the Alabama State Capitol. Government work and surveys, where he could be surrounded by friends, were always attractive to Mr. Williamson, and hence, in 1891, he went with the United States Engineers on river and harbor work again. He was engaged under Major F. A. Mahan as United States Civil Assistant Engineer on the Coosa River, where he remained until the war with Spain began, when he was transferred to Pensacola, Fla., and had charge of work upon fortifications and harbor defences. While upon this work the climate and overwork brought on an attack of dysentery which resulted in his death.

Mr. Williamson was descended from English ancestors through a line of distinguished Virginians. He was an engineer of more than usual ability, both in theory and in practice, a better mathematician than most engineers of ante-bellum college training, precise in everything and an excellent draftsman. He was devoted to his profession and untiring in his duties, a man of varied experience, and, in the words of one who was associated with him, "so honorable in every relation of life, particularly professionally, that there would be no need of a code of ethics in the profession, were all its members but one-half so far above temptation as was Captain Williamson." Another engineer writes of him as "one of the most open-hearted and lovable of men, honest and true to a fault, and, withal, a man of uncompromising principle." He married Miss Bettie Cralle, of Huntington, W. Va., October 10th, 1873, and she and seven children survive him.

Mr. Williamson was elected a Member of the American Society of Civil Engineers April 4th, 1883.

WILLIAM ROBERTS MICHIE, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 2D, 1899.

This young engineer, cut off in his prime, was born at Yanktown, McDowell County, N. C., September 22d, 1866. His father, Peter S. Michie, now Professor of Natural and Experimental Philosophy at the United States Military Academy at West Point, had served with distinction through the Civil War, had risen to the rank of General of Volunteers, and was at that time in the regular army. As the engineer officer of the Army of the James, Gen. Michie had rendered distinguished services, so that the son came by his taste for engineering in a manner almost hereditary. It was the desire of the parents that the young man should be educated at the United States Military Academy, but, owing to a slight trouble with his eyes, he was sent instead to Dartmouth College, where he was graduated as a civil engineer, in April, 1889, from the Thayer School of Civil Engineering. Professor Robert Fletcher, the head of the Thayer school at that time, was a graduate of West Point, so that, although Mr. Michie was denied admission to the Military Academy on account of a disability which was never perceptible, he nevertheless obtained an education modeled very much on the West Point lines. He showed his ability at once on graduation to such an extent that after a brief service on the St. Louis and San Francisco Railway he rose to the position of assistant engineer. He had charge, for that railway, of the construction of a pier and one span of the Red River Bridge, and made surveys for wing dams to change the current of the Red River.

He entered the service of the Pennsylvania Railroad Company, April 20th, 1889, upon the recommendation of Mr. Frank Thomson, and had occupied the following positions on this celebrated railroad: 1889, chainman on construction of the Ardara Bridge, also at Viaduct and South Fork Bridges until August, when he became rodman on the construction of the Wall repair shops; 1890, in July, Acting Assistant Engineer in the construction of a new yard at Altoona; in October promoted to Assistant Engineer, Altoona yards, surveys for South Fork Railroad and part of its construction; construction of Wall yard and round houses, a large railroad yard near Pittsburg; construction of the Sang Hollow extension, approximately fifteen miles; 1895, construction of third and fourth-track work on the Pittsburg Division and at various points on the Johnstown Branch; 1896, completing the Wilmerding Branch, and on the new line at Nineveh and the Radebaugh Tunnel; 1897, construction work on the Pittsburg Division, change of line and additional tracks at various points; 1898, construction work, additional track-work in Johnstown and vicinity and the Portage Tunnel; 1899, resigned January 10th.

* Memoir prepared by William H. Wiley, M. Am. Soc. C. E.

To show what rare capacity he exhibited at the beginning of his career some details are given: In November, 1890, within six weeks' time, he made the surveys for the extension from South Fork to Dunlo, and the line was afterward built on his location. The change of line from Portage to Tilly was also constructed practically in accordance with Mr. Michie's surveys, made in 1891. In that year, also, Mr. Michie had charge of much important work on the Pittsburg Division. Entering upon the construction of the extensive Wall yards, in the early stages of the work, he carried it to completion in December, 1892, and built at the same time a 44-stall roundhouse. Under his direction the line between Turtle Creek and Brinton was changed, and the Wilmerding Branch Railroad was constructed.

After the Johnstown flood Mr. Michie was engaged in the reconstruction of the track, and was inspector in charge of the construction of the celebrated Viaduct Bridge near South Fork, Pa. This structure, which is notable as one of the largest arch bridges in the United States, has two 80-ft. spans and is over 100 ft. in height. He also had charge of the extensions to the large arch bridges over the Juniata River; five spans of 60 ft. each. In addition to the record of service given as above by years, Mr. Michie was in charge of numerous important surveys for changes of the line and branches on the Philadelphia and Erie, and on the Northern Central Railroads, as well as on the Baltimore and Potomac Railroad, near Washington, D. C. He had full charge of building the Radebaugh Tunnel, 2 100 ft. long, near Greensburg, Pa., and was so successful with this work that he was detailed to superintend the rebuilding of the celebrated Portage Tunnel at Gallitzin, Pa. This was the last construction work he undertook for the railroad company, as he resigned his position in January, 1899, to accept an advantageous appointment in Cuba. Mr. Michie was quite familiar with the uses of steam and with steam engines and machinery generally, and could handle a locomotive, apparently, as well as the engine-runner.

This, in brief form, is the record of an engineer who, had he lived, would have attained great distinction in his profession. The writer knew him when he was a boy, and he possessed such lovable traits of character that he drew everyone toward him. As a man he justified all these early characteristics and was one of the most popular men on the Pennsylvania Railroad. For the preparation of this memoir there were available not only many personal letters from his associates and from those under his immediate charge, but also newspaper extracts containing accounts of Mr. Michie and his work, and all of these expressed the sentiment of a personal loss sustained by the writers.

Mr. Michie was elected a Junior of the American Society of Civil Engineers, February 28th, 1893, and an Associate Member, October 7th, 1896.

WATERMAN STONE, Assoc. Am. Soc. C. E.*

DIED MARCH 30TH, 1896.

Waterman Stone was the eldest son of Lemuel M. E. Stone, one of the engineers engaged in early railroad work in New England, and was born in Cumberland, R. I., March 10th, 1847. His engineering training was under his father's direction. In 1871 he succeeded his father as Superintendent and Engineer of the Providence, Warren and Bristol Railroad, and during the eighteen years he held this position he designed and supervised all the improvements made on the road. The most notable of these was the construction of a curve of 211 ft. radius in the main line, just outside the City of Providence, R. I., to enable trains to enter and leave the city head on. A bluff 130 ft. high rose from the East Providence shore of the Blackstone River at the point where the railroad crossed, and since the construction of the road, trains had been run on a Y-track above the bridge, and were pushed in and out of the city. Mr. Stone put in a 27° 20'-curve between the river s edge and the bluff. A modified Fairlie type of locomotive was built to haul trains around this curve and standard eight-wheel American type locomotives were also run over it. The forward pair of driving-wheels of such engines was equipped with blank tires with a width of tread of 7 ins., and third and fourth rails were laid on the curve for the blank tires.

On the absorption of the Providence, Warren and Bristol Railroad by the Old Colony system, Mr. Stone removed to Kansas City, Mo., where he became Superintendent of the Inter-State Rapid Transit Railroad. This was a cable railway, but during the four years of his service electric traction was installed. Returning to the East, Mr. Stone began practice as a consulting engineer, with especial reference to electric railroads. He had built several roads and was engaged upon others at the time of his death.

Mr. Stone was a man of sterling character, and his kindly disposition and love of fair play made him as popular with his employees as with his more general acquaintance. Of mathematical tendency, he was an earnest student in fields not bounded by his professional practice. He was elected an Associate of the American Society of Civil Engineers, December 1st, 1886. While in the West he was Secretary and Treasurer of the Engineers' Club of Kansas City, and he was the first Secretary of the American Society of Railroad Superintendents.

* Memoir prepared by the Secretary from information furnished through E. P. Dawley, M. Am. Soc. C. E., and from papers on file.